DESIGN METHODOLOGY

DESIGN METHODOLOGY
SECTION 5

CONTENTS

SYMBOLS USED IN THIS SECTION

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DESIGN METHODOLOGY

DISCLAIMER

The information in this manual is provided as a guide to assist you with your design and in writing your own specifications.

Installation conditions, including soil and structure conditions, vary widely from location to location and from point to point on a site.

Independent engineering analysis and consulting state and local building codes and authorities should be conducted prior to any installation to ascertain and verify compliance to relevant rules, regulations and requirements.

Hubbell Power Systems, Inc., shall not be responsible for, or liable to you and/or your customers for the adoption, revision, implementation, use or misuse of this information. Hubbell, Inc., takes great pride and has every confidence in its network of installing contractors and dealers.

Hubbell Power Systems, Inc., does NOT warrant the work of its dealers/installing contractors in the installation of CHANCE® Civil Construction foundation support products.

5.1 ATLAS RESISTANCE® PIER CAPACITY

ATLAS RESISTANCE® Piers develop their capacity primarily through end bearing. The current accepted state of the art practice is for ATLAS RESISTANCE® Piers to be installed to a preset performance design criterion. The development of a theoretical capacity model is under study. Current and planned research projects and studies should provide meaningful data for the development of this model in the future.

In general, the tip of the ATLAS RESISTANCE® Pier should be embedded in cohesionless soils with Standard Penetration Test (SPT) "N" values above the 30 to 35 range and in cohesive soils with SPT "N" values above the 35 to 40 range. The ATLAS RESISTANCE® Pier will provide foundation underpinning support in end-bearing when positioned into these SPT "N" value ranges based on past installation experience. See Figures 5-1 and 5-2 for assumed failure patterns under a pile tip in dense sand.

The ATLAS RESISTANCE® Pier is a manufactured, two-stage product designed specifically to produce structural support strength. First, the pier pipe is driven to a firm-bearing stratum then the lift equipment is combined with a manifold system to lift the structure. The ATLAS RESISTANCE® Pier System procedure provides measured support strength. ATLAS RESISTANCE® Piers are spaced at adequate centers where each pier is driven to a suitable stratum and then tested to a force greater than required to lift the structure. ***This procedure effectively load tests each pier prior to lift and provides a measured Factor of Safety (FS) on each pier at lift.***

Performance Design Criterion

The following guidelines are intended to serve as a basis for the selection and installation of a proper ATLAS RESISTANCE® Pier.

- Pier Spacing: The required working load per pier is calculated based on the dead loads and live loads and the ability of the existing foundation to span between the proposed pier locations.

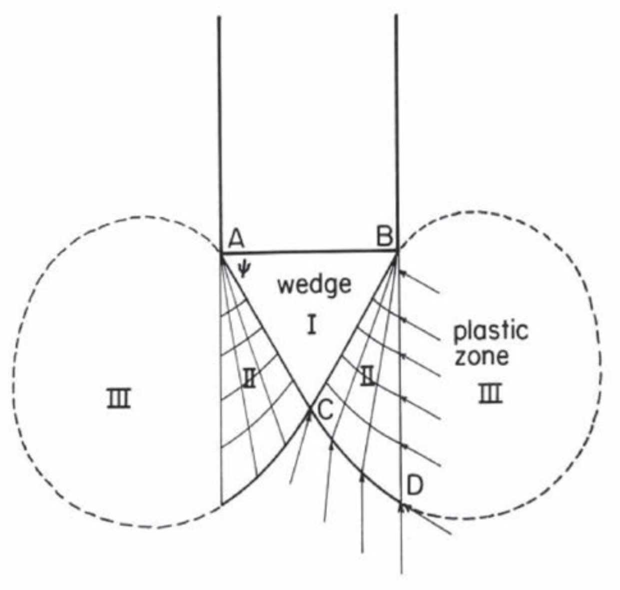

Figure 5-1 Assumed Failure Pattern Under Pile Point

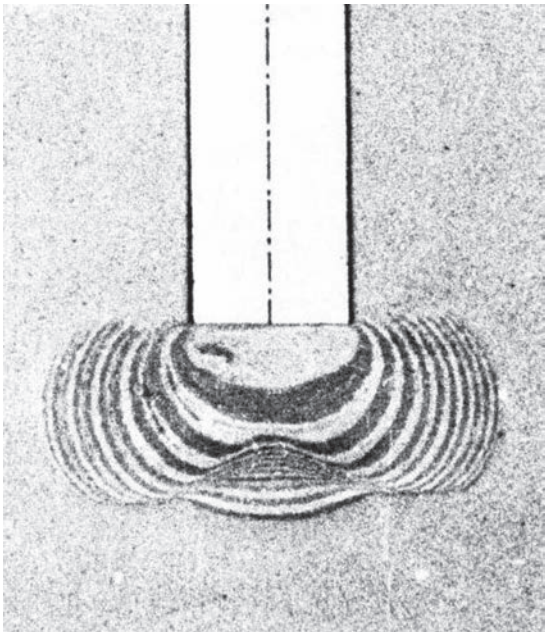

Figure 5-2 Failure Pattern Under Pile Point in Dense Sand

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	P	=	DL + LL + SL + W
	P_w	=	(x) x (P)
	P	=	Line load on footing
	P_w	=	Pier working load
where	DL	=	Dead load
	LL	=	Live load
	SL	=	Snow load
	W	=	Soil load
	x	=	Selected pier spacing

- Select Factor of Safety: Hubbell Power Systems, Inc. recommends a minimum Factor of Safety (FS_h) for mechanical strength of the hardware of 2.0.

	FS_h	=	2.0 (may be varied based on engineering judgment)
where	$R_{w\ ULT}$	=	$P_w \times FS_h$
	$R_{w\ ULT}$	=	Minimum ultimate hardware strength based on structural weight

- Select a Pier System with an adequate minimum ultimate strength rating.

	$R_{h\ ULT}$	$\geq$	$2 \times P_w$
where	$R_{h\ ULT}$	=	Minimum ultimate hardware strength based on the published strength rating found in Section 7 of this Technical Design Manual

- Check the maximum pier spacing (x_{MAX}) based upon the selected hardware capacity.

x_{MAX}	=	$(R_{h\ ULT}) / (FS_h) \times (P)$ (wall and footing must be structurally capable of spanning this distance)
x	$\leq$	x_{MAX}

- Proof Load: ATLAS RESISTANCE® Piers are installed using a two-step process as noted above. First, the ATLAS RESISTANCE® Pier is driven to a firm bearing stratum. The resistance force applied during this step is called the Proof Load (R_p). Hubbell Power Systems, Inc. recommends a minimum Factor of Safety[1] (FS_P) of 1.5 at installation unless structural lift occurs first.

$$R_p = (FS_P) \times (P_W)$$
$$R_p = 1.5 \times (P_W)$$
$$R_{h\ MAX} = (R_{h\ ULT} / FS_h) \times 1.65$$
$$R_{h\ MAX} = (R_{h\ ULT} / 2.0) \times 1.65$$
$$R_p < R_{h\ MAX}$$

where $R_{h\ MAX}$ = Maximum installation force based on hardware ultimate capacity[2]

[1] Experience has shown that in most cases the footing and stem wall foundation system that will withstand a given long term working load will withstand a pier installation force of up to 1.5 times that long term working load. If footing damage occurs during installation, the free span between piers ($L_{P\ MAX}$) may be excessive.
[2] It is recommended that $R_{h\ MAX}$ not exceed $(R_{h\ ULT} / 2) \times 1.65$ during installation without engineering approval.

Additional Notes:

Current practice by Hubbell Power Systems, Inc. is to limit the unsupported pier pipe exposure to a maximum of 2 feet at the published working loads for the standard pier systems. The soil must have a SPT "N" of greater than 4. The pier pipe must be sleeved for pier pipe exposures greater than 2 feet and up to 6 feet and/or through the depths where the SPT value "N" is 4 or less. Sleeve must extend at least 36" beyond the unsupported exposure and/or the area of weak soil. If the anticipated lift is to exceed 4", then the ATLAS RESISTANCE® Continuous Lift Pier System should be used.

ATLAS RESISTANCE® Piers can be located as close as 12" (305 mm) between adjacent piers to develop a "cluster" of load bearing elements.

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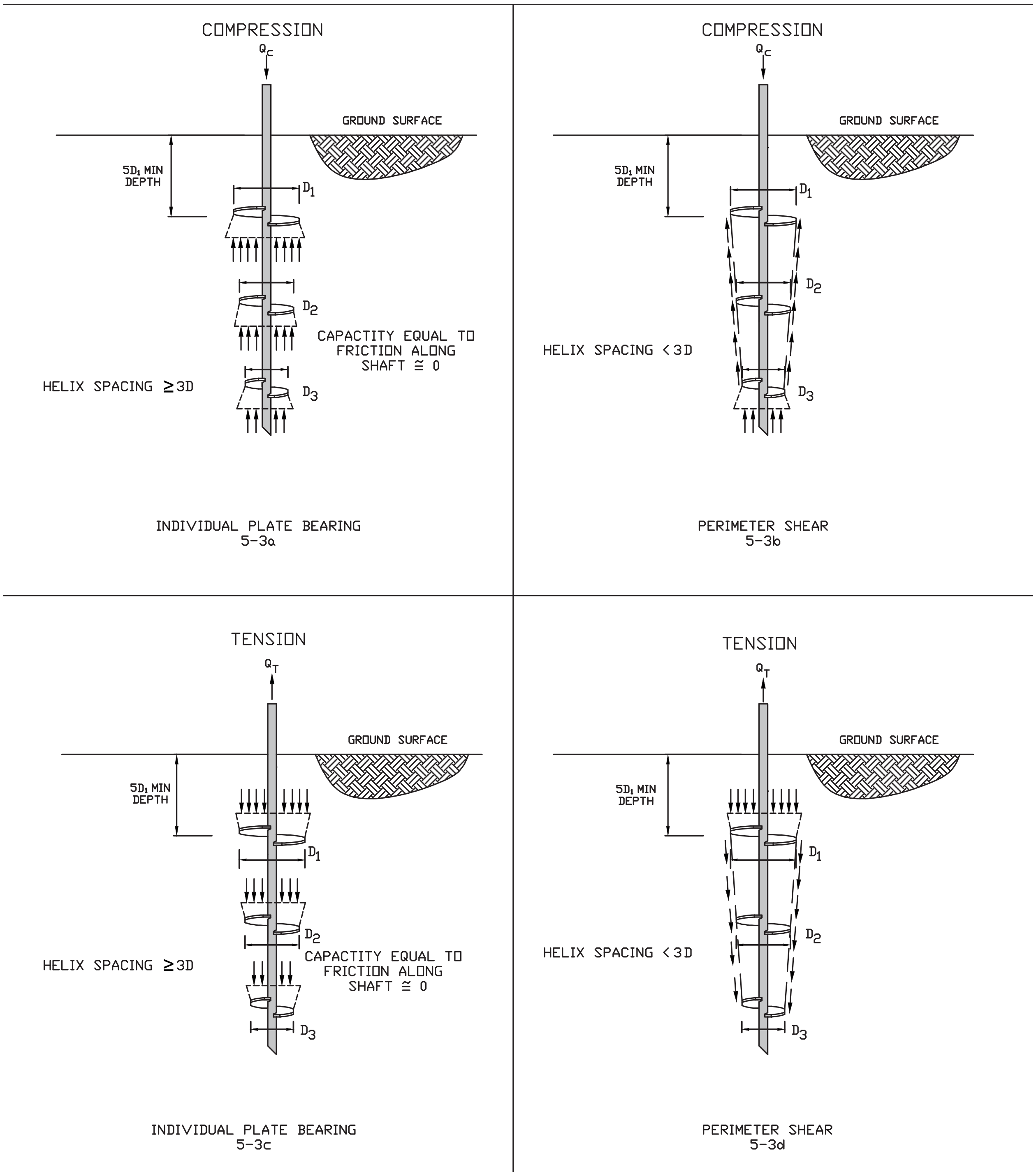
5.2 CHANCE® HELICAL PILE/ANCHOR ULTIMATE BEARING CAPACITY

The capacity of a helical pile/anchor is dependent on the strength of the soil, the projected area of the helix plate(s), and the depth of the helix plate(s) below grade. The soil strength can be evaluated by use of various field and lab techniques. The projected area is controlled by the size and number of helix plates. Helical anchors and screw piles may be used for a variety of applications involving both tension loading (helical anchors) and compression loading (screw piles or helical piles). Screw piles and helical anchors are generally classified as either "shallow" or "deep" depending on the depth of installation of the top helix below the ground surface, usually with respect to the helix diameter. There are some situations in which the installation may be considered partway between "shallow" and "deep", or "intermediate". In this Manual, only design procedures for "shallow" and "deep" installations will be described. Table 1 gives a summary of the most common design situations involving screw-piles and helical anchors that might be encountered. Note that the use of "shallow" multi-helix anchors for either compression or tension loading is not a typical application and is not covered in this Technical Design Manual.

The dividing line between shallow and deep foundations has been reported by various researchers to be between three and eight times the foundation diameter. To avoid problems with shallow installations, the minimum recommended embedment depth of helical piles and anchors is five helix diameters (5D). The 5D depth is the vertical distance from the surface to the top-most helix. Whenever a CHANCE® Helical Pile/Anchor is considered for a project, it should be applied as a deep foundation for the following reasons:

1. A deep bearing plate provides an increased ultimate capacity both in uplift and compression.
2. The failure at ultimate capacity will be progressive with no sudden decrease in load resistance after the ultimate capacity has been achieved.

The approach taken herein for single-helix piles/anchors assumes that the soil failure mechanism will follow the theory of general bearing capacity failure. For multi-helix helical piles and anchors, two possible modes of



Individual Bearing and Perimerter Shear Models for Helical Piles with Slender Shafts
Figure 5-3

failure are considered in design, depending on the relative spacing of the helix plates. For wide helix spacing (s/B ≥ 3), the Individual Plate Bearing Method is used; for close helix spacing (s/B < 3), the Perimeter Shear Method is used. These two methods are illustrated in Figures 5-3a & c (Individual Plate Bearing) and 5-3b & d (Perimeter Shear). With Individual Plate Bearing, the helix capacity is determined by calculating the unit bearing capacity of the soil at each helix and then multiplying the result by the individual helix's projected area. Friction along the central shaft is typically not used to determine capacity, but may be included when the central shaft is round, as will be discussed later in this section. The Individual Plate Bearing Method assumes that load capacity will be developed simultaneously and independently by each helix; i.e. no interaction between helix plates. The Perimeter Shear Method assumes that because of the close helix spacing, a prism

Table 5-1 Typical Design Situations for Single-Helix and Multi-Helix Screw-Piles and Helical Anchors

Single-Helix				Multi-Helix			
Failure Condition				Failure Condition			
Shallow		Deep		Shallow		Deep	
C	T	C	T	C	T	C	T
Clay	Clay	Clay	Clay	N/A	N/A	Clay	Clay
Sand	Sand	Sand	Sand	N/A	N/A	Sand	Sand
Mixed Soils	Mixed Soils	Mixed Soils	Mixed Soils	N/A	N/A	Mixed Soils	Mixed Soils

C = Compression T = Tension

of soil will develop between the helix plates and failure in this zone occurs along a plane as shown in Figure 5-3b & d. In reality, the Perimeter Shear Method includes both plate bearing and perimeter shear failure as illustrated.

The following is Terzaghi's general bearing capacity equation, which allows determination of the ultimate capacity of the soil. This equation and its use will be discussed in this section, as it forms the basis of determining helix capacity in soil.

$$Q_{ult} = A_h \left(cN_c + q'N_q + 0.5\, \gamma' BN\gamma \right)$$

where

Q_{ult} = Ultimate capacity of the soil
A_h = Projected helix area
c = Soil cohesion
q' = Effective overburden pressure
B = Footing width (base width)
γ' = Effective unit weight of the soil
and Nc, Nq, and Nγ are bearing capacity factors

Terzaghi's Bearing Capacity Factors are shown in the Table 5-2.

Table 5-2. Terzaghi's Shallow Foundation Bearing Capacity Factors [from and Bowles (1988) and ASCE (1993a)]

ϕ'	Nc	$N\gamma$	Nq
0	5.7	0.0	1.0
10	9.6	1.2	2.7
12	10.8	1.7	3.3
14	12.1	2.3	4.0
16	13.7	3.0	4.9
18	15.5	3.9	6.0
20	17.7	4.9	7.4
22	20.3	5.8	9.2
24	23.4	7.8	11.4
26	27.1	11.7	14.2
28	31.6	15.7	17.8
30	37.2	19.7	22.5
32	44.0	27.9	28.5
34	52.6	36.0	36.5
36	63.5	52.0	47.2
38	77.5	80.0	61.5
40	95.7	100.4	81.3
42	119.7	180.0	108.7
44	151.9	257.0	147.7
46	196.2	420.0	204.2
48	258.3	780.1	287.8

Following is quoted from Bowles (1988) concerning the use of Equation 5-6 for deep foundations where the various terms of the bearing capacity equation are distinguished.

"1. The cohesion term predominates in cohesive soil.

2. The depth term ($q'N_q$) predominates in cohesionless soil. Only a small D (vertical depth to footing or helix plate increases Q_{ult} substantially.

3. The base width term $0.5\gamma'BN\gamma$ provides some increase in bearing capacity for both cohesive and cohesionless soils. In cases where B is less than about 2 feet (0.61 m), this term could be neglected with little error."

The base width term of the bearing capacity equation is not used when dealing with helical anchors/piles because, as Bowles indicates, the resulting value of that term is quite small. The effective overburden pressure (q', of consequence for cohesionless soils) is the product of depth and the effective unit weight of the soil. The water table location may cause a reduction in the soil bearing capacity. The effective unit weight of the soil is its in-situ unit weight when it is above the water table. However, the effective unit weight of soil <u>below</u> the water table is its in-situ unit weight less the unit weight of water.

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Notes on use of Terzaghi's Bearing Capacity equation:

1. Because helix plates are generally round, Terzaghi's adjustment for round footings is sometimes used for compression loading:

   a. $Q_H = A_H(1.3c'N_C + q'N_q + 0.6\gamma' BN\gamma)$

2. Because B is considered very small for screw-piles and helical anchors, relative to most concrete footings, most engineers choose to ignore the term $0.5\gamma' BN\gamma$ in design.
3. In saturated clays under compression loading, Skempton's (1951) Bearing Capacity Factor for shallow round helical plates can also be used:

   a. $N_C = 6.0(1 + 0.2D/B) \leq 9.0$
4. The unit weight of the soil is the total (wet) unit weight if the helical plate (s) is above the water table and the buoyant unit weight if the helical plate(s) is below the water table.
5. For saturated clay soils, $N_q = 1.0$; For sands, N_q is a function of the friction angle, φ'.
6. For square-shaft anchors/piles, the shaft resistance is generally ignored. For round shaft piles/anchors there may be a component of shaft resistance that contributes to capacity depending on the configuration of connections between extension sections.
7. In all cases, for both compression and tension loading, the upper limit of capacity is governed by the mechanical strength of the pile/anchor as provided by the manufacturer. See Section 7 of this Manual for mechanical strength ratings of CHANCE® Helical Piles/Anchors.

Concern can develop when a helical pile/anchor installation is terminated in sand above the water table with the likelihood that the water table will rise with time to be above the helix plates. In this situation, the helical pile/anchor lead section configuration and depth should be determined with the water at its highest anticipated level. Then the capacity of the same helical-pile/anchor should be determined in the same soil with the water level below the helical pile/anchor, which will typically produce higher load capacities and a more difficult installation, i.e., it will require more installation torque. It is sometimes the case that a larger helical pile/anchor product series, i.e., one with greater torque capacity, must be used in order to facilitate installation into the dry conditions.

5.2.1 Single-Helix Screw-Piles and Helical Anchors – Shallow Installation

5.2.1.1 Compression Loading (Shallow Single-Helix)

A shallow installation, like a shallow foundation, is one in which the ratio of depth (D) of the helix to diameter (B) of the helix is less than or equal to about 5, i.e., $D/B \leq 5$. In this case, the design is very analogous to compression loading of a shallow foundation.

5.2.1.1.a Saturated Clays φ′ = 0; c > 0

In saturated clays with φ′ = 0, the term Nγ = 0 and Nq = 1.0. The bearing capacity equation becomes:

$$Q_H = A_H(cN_C + \gamma' D) \qquad \textbf{Equation 5-9}$$

where:

Q_H = Ultimate Bearing Capacity
A_H = Projected Helix Area
c = "cohesion"; for φ′ = 0; c = undrained shear strength = s_u
N_C = Bearing Capacity Factor for φ′ = 0; for round plates $N_C = 6.0(1 + 0.2D/B) \leq 9$
γ′ = effective unit weight of soil above screw-pile
D = Depth

Note: The term γ′D is sometimes ignored because it is very small.

5.2.1.1.b Sands φ′ > 0; c′= 0

In clean sands with zero cohesion, the cohesion term of the bearing capacity equation drops out and only two terms remain:

$$Q_H = A_H(q'N_q + 0.5\gamma' BN\gamma) \qquad \textbf{Equation 5-10}$$

where:

q′ = effective surcharge (overburden pressure) = γ′D
N_q and Nγ are evaluated from the Table of Bearing Capacity Factors

Note: The term 0.5γ′BNγ is typically ignored for helical piles because the helix plate is small

5.2.1.1.c Mixed Soils φ′ > 0; c′ > 0

Many soils, such as mixed-grain silty sands, sandy silts, clayey sands, etc., have both a frictional and cohesive component of strength. In these cases, the bearing capacity equation includes all three terms:

$$Q_H = A_H(c'N_C + q'N_q + 0.5\gamma' BN\gamma) \qquad \textbf{Equation 5-11}$$

Note: The term 0.5γ′BNγ is typically ignored for helical piles because the helix plate is small.

5.2.1.2 Tension Loading - Axial Uplift (Shallow Single Helix)

Under tension loading in axial uplift, the behavior of a shallow single-helix helical anchor is currently approached more-or-less as an "inverse" bearing capacity problem and the concern is for the failure surface to reach the ground surface, producing "breakout" of the helical plate. Helical anchors should not be installed at vertical depths less than 5 ft. for tension loading. The design approach is similar to that under compression loading, except that instead of using a Bearing Capacity Factor, N_C, a Breakout Factor, F_C, is used.

5.2.1.2.a Saturated Clays $\varphi' = 0$; $c > 0$

Test results and analytical studies indicate that the Breakout Factor for saturated clays in undrained loading varies as a function of the Relative Embedment of the plate, i.e., D/B. This is much like the transition of shallow to deep foundation behavior under compression loading. Table 5-3 shows the variation in F_C vs. D/B for circular plates. This figure (from Das (1990) shows that $F_C = 1.2(D/B) \leq 9$, so that at $D/B > 7.5$, $F_C = 9$ (i.e., the transition from shallow to deep behavior under tension in clays occurs at about $D/B > 7.5$). In this case, the ultimate uplift capacity is similar to Equation 5-9 and is given as:

$$Q_{HU} = A_H(cF_C + \gamma' D)$$

where:

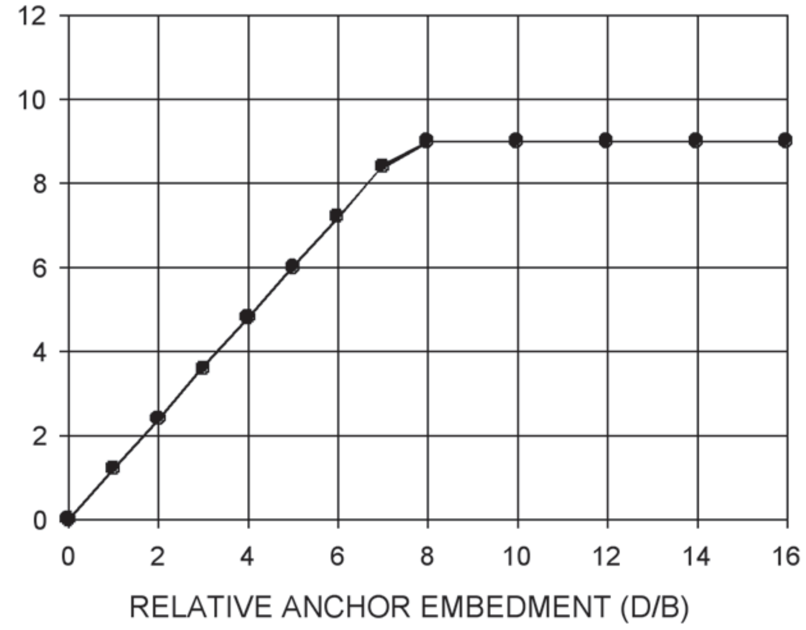
Q_{HU} = Ultimate Uplift Capacity
c = "cohesion"; for $\varphi' = 0$ c = undrained shear strength = s_u

F_C = Breakout Factor for $\varphi' = 0$; $F_C = 1.2(D/B) \leq 9$
γ' = effective unit weight of soil above helical anchor plate
D = Depth

Note: The term $\gamma'D$ is sometimes ignored because it is very small.

In some situations the undrained shear strength of clays under tension loading may be reduced to account for some disturbance effects of the clay above the helical plate but this is a matter of engineering judgment.

Table 5-3 Variation in Uplift Breakout Factor for Shallow Single-Helix Anchors in Clay



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5.2.1.2.b Sands $\varphi' > 0$; $c' = 0$

In sands the uplift behavior of shallow (generally $D/B \leq 5$) single-helix anchors develops a failure zone that looks similar to an inverted truncated cone. The failure is assumed to take place by the perimeter shear acting along this failure surface, which is inclined from the vertical at an angle of about $\varphi'/2$, as shown in Figure 5.4, and also includes the mass of the soil within the truncated cone. The Ultimate Uplift Capacity is calculated from:

$$Q_{HU} = W_S + \pi\gamma K_0(\tan\varphi')(\cos^2\varphi'/2)\,[(BD^2/2) + (D^3\tan\varphi'/2)/3)] \qquad \textbf{Equation 5-13}$$

where:

W_S = Mass of Soil in Truncated Cone = γV
γ = Total (wet) Unit Weight
V = Volume of Truncated Cone
K_0 = At-Rest Lateral Earth Pressure Coefficient
B= helix diameter
D = vertical plate depth

The volume of the truncated cone is determined from:

$$V = [\pi D/3][B^2 + (B + 2D\tan\varphi'/2)^2 + (B)(B + 2D\tan\varphi'/2)] \qquad \textbf{Equation 5-14}$$

Values of the at-rest lateral earth pressure coefficient for sands can reasonably be taken as:

$K_0 = 1 - \sin\varphi'$

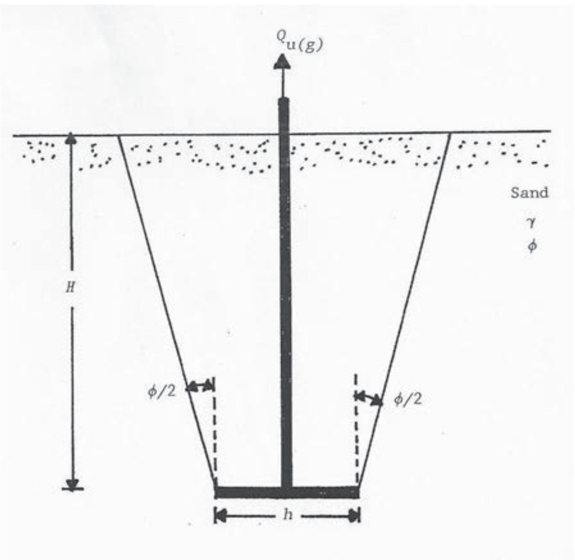

Figure 5-4 Proposed Failure Mechanism for Shallow Single-Helix Anchors in Dense Sand.

5.2.1.2.c Mixed Soils $\varphi' > 0$; $c' = 0$

In mixed soils with both frictional and cohesive components of shear strength, there is an added resisting force in uplift for shallow installations above the value given by Equation 5-13. This added component results from cohesion acting along the surface of the truncated cone failure zone between the helical plate and the ground surface so that an additional term may be added to Equation 5-13 giving:

$$Q_{HU} = W_S + \pi\gamma K_0(\tan\varphi')(\cos^2\varphi'/2)\,[(BD^2/2) + (D^3\tan\varphi'/2)/3)] + (c)(A_C) \qquad \textbf{Equation 5-15}$$

where:

A_C = Surface Area of Truncated Cone

The surface area of a truncated cone can be obtained from:

$$A_C = \pi[(R^2 + r^2) + [(R^2 - r^2) + (D(R + r))^2]^{0.5}] \qquad \textbf{Equation 5-16}$$

where:

r = Radius of Helical Plate = B/2
R = Radius of Cone Failure Surface at the Ground Surface = B/2 + (D)tan($\varphi'/2$)

The additional component of uplift resulting from soil cohesion, is sometimes ignored since soil cohesion is often lost from water infiltration or rising water table.

5.2.2 Single-Helix Screw-Piles and Screw-Anchors – Deep Installation

Deep installations of screw-piles and helical anchors are generally more common than shallow installations, provided there is sufficient soil depth to actually perform the installation. The reason is simply that higher load capacities are generally developed from a deeper installation in the same soil so it makes more sense economically to go for a deep installation when possible. Figure 5.5 below demonstrates the single-helix plate capacity model, where the soil failure mechanism will follow the theory of general bearing plate capacity. Compression capacity is mobilized from soil below the helix plate and tension capacity from soil above the helix plate.

5.2.2.1 Compression Loading (Deep Single-Helix)

A deep installation, like a deep foundation, is one in which the ratio of depth (D) of the helix to diameter (B) of the helix is greater than 5 - 7, i.e., D/B > 5 - 7. In this case, the design is very analogous to compression loading of deep end bearing foundation.

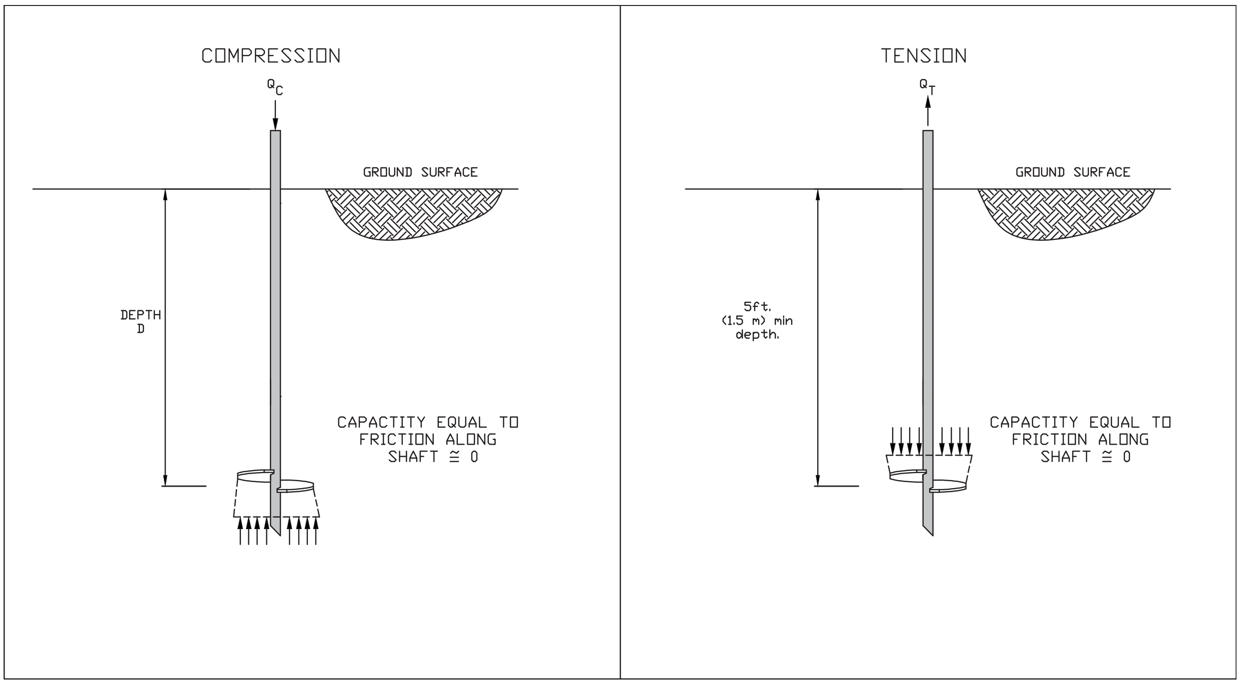

Figure 5-5 Single-Helix Plate Bearing Capacity Model – Helical Piles with Slender Shafts

5.2.2.1.a Saturated Clays φ' = 0; c' > 0

Under compression loading, the ultimate capacity of a single-helix screw-pile in clay is calculated from Equation 5-9 as:

$Q_H = A_H[(N_C)(s_u) + \gamma' D]]$

where:

N_C = Bearing Capacity Factor for Deep Failure = 9

Which gives:

$Q_H = A_H[(9)(s_u) + \gamma' D]$ **Equation 5-17**

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5.2.2.1.b Sands φ' > 0; c' = 0

For clean, saturated sands, the "cohesion" is normally taken as zero, reducing the ultimate capacity, as in Equation 5-10, to:

$$Q_H = A_H(q'N_q + 0.5\gamma' BN\gamma)$$

Even in moist sands or sand with a small amount of fines that may give some "cohesion", this is usually ignored. Because the area of the plate is small, the contribution of the "width" term to ultimate capacity is also very small and the width term is often ignored leaving:

$$Q_H = A_H(q'N_q) \qquad \textbf{Equation 5-18}$$

For deep installations, the bearing capacity factor Nq is usually obtained from values used for determining the end bearing capacity for deep pile foundations, which is different than the values used for shallow foundations. There are a number of recommendations for Nq available in foundation engineering textbooks as shown in Figure 5-6. The difference in Nq values shown in Figure 5-6 is largely related to the assumptions used in the failure mechanism. Figure 5-7 gives a reasonable chart of Nq values as a function of the friction angle of the soil, φ', that may be used for screw-piles and helical anchors. The value of Nq in Figure 5-7 is obtained from:

$$Nq = 0.5\ (12 \times \varphi')^{\varphi'/54} \qquad \textbf{Equation 5-19}$$

Note: In some sands, the unit end bearing capacity of deep foundations may reach a limiting value. The point at which this occurs is generally termed the "critical depth". Critical depth is defined as the depth at which effective vertical stress, a.k.a. overburden pressure, will not increase with depth. Critical depth is not specifically defined for screw-piles and helical anchors, but engineers often use it with deep installation in saturated sands.

5.2.2.1.c Mixed Soils φ' > 0; c' > 0

The ultimate capacity of a deep single-helix screw-pile in mixed-grain soils can be taken from traditional bearing capacity theory using Equation 5-11:

$$Q_H = A_H(cN_c + q'N_q + 0.5\gamma BN\gamma)$$

Note: The term $0.5\gamma' BN\gamma$ is typically ignored for helical piles because the helix plate is small.

5,2.2.2 Tension Loading –Axial Uplift (Deep Single-Helix)

5.2.2.2.a Saturated Clays φ' = 0; c' > 0

Under tension loading, the ultimate capacity of a single-helix screw-anchor in clay the ultimate capacity is calculated using the same approach given in Section 5.2.2.1.a. In some cases a reduction may be made in the undrained shear strength to account for soil disturbance above the helical plate as a result of installation, depending on the Sensitivity of the clay. Also, as previously noted in Section 5.2.1.2.a, for a deep installation (D/B > 7.5) the Breakout Factor, F_C has a default value of 9. The bearing capacity equation becomes:

$$Q_{HU} = A_H[(9)s_u + \gamma' D]$$

5.2.2.2.b Sands φ' > 0; c' = 0

In sands, the tension capacity of a helical anchor is generally assumed to be equal to the compression capacity provided that the soil above the helix is the same as the soil below the helix in a zone of about 3 helix diameters. Again, for clean, saturated sands, the "cohesion" is normally taken as zero, reducing the ultimate capacity to:

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$$Q_H = A_H(q'N_q + 0.5\gamma' B N\gamma)$$

Also, because the area of the plate is small, the contribution of the "width" term to ultimate capacity is also very small and the width term is often ignored leaving:

$$Q_H = A_H(q'N_q)$$

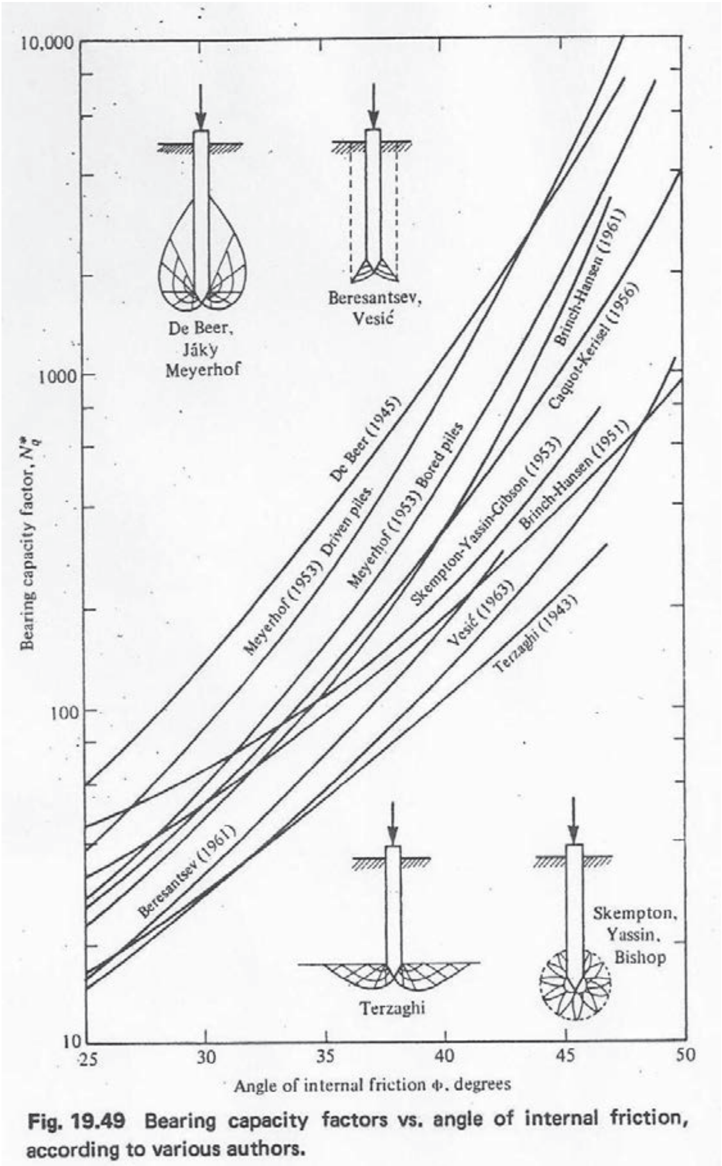

Figure 5-6 Reported Values of Nq for Deep Foundations in Sands [from Winterkorn & Fang (1983)].

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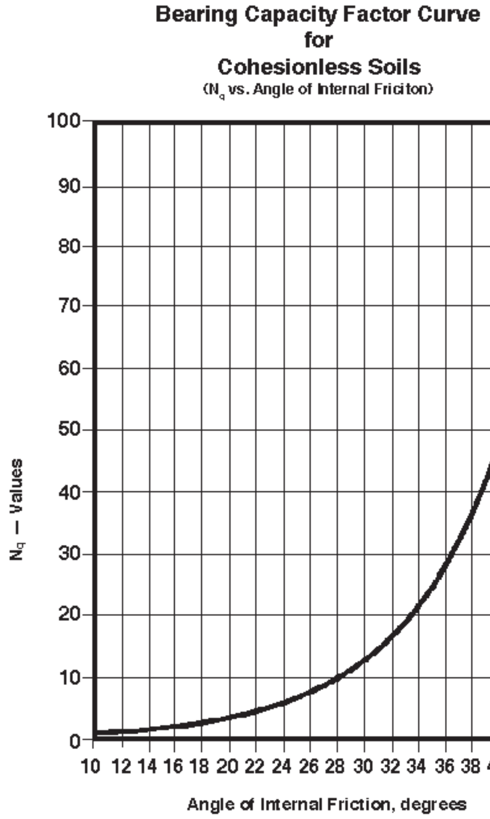

Figure 5-7 Recommended Bearing Capacity Factor Nq for Deep Screw-Piles and Helical Anchors in Sand.

5.2.2.2.c Mixed Soils $\varphi' > 0$; $c' > 0$

The ultimate capacity of a deep screw-pile in mixed-grain soils can be taken from traditional bearing capacity theory using Equation 5-11:

$$Q_H = A_H(cNc + q'Nq + 0.5\gamma BN\gamma)$$

Note: The term $0.5\gamma' BN\gamma$ is typically ignored for helical piles because the helix plate is small.

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5.2.3 Multi-Helix Screw-Piles and Screw-Anchors – Deep Installation

The ultimate capacity of deep multi-helix screw-piles and screw-anchors depends on the geometry of the helical section, namely the size and number of helical plates and the spacing between the plates. As shown in Figure 5-3b and 5-3d, if the spacing of helix plates is close, the capacity is developed from a zone of failure between the helical plates and from end bearing from the end helix plate (either the lowest plate for compression loading or the top helix plate for tension loading), i.e., the helix plates interact with each other. If the spacing of the helix plates is sufficiently large, the capacity is taken as the sum of the capacity developed from the individual helix plates, i.e., there is no interaction between helix plates. Also, there is no capacity taken along the shaft between the helix plates.

In the U.S., most manufacturers of screw-piles and helical anchors produce elements with a standard helix spacing of 3 times the helix diameter. This spacing was originally used by CHANCE® over 30 years ago and is assumed to allow individual helix plates to develop full capacity with no interaction between helix plates and the total capacity is taken as the sum of the capacities from each plate as shown in Figure 5-3a and 5-3c. Most CHANCE® Screw-Piles and Helical Anchors use inter-helix spacing that is based on the diameter of the lower helix. For example, the distance between a 10 inch (254 mm) and a 12 inch (305 mm) helix is three times the diameter of the lower helix, or 10 x 3 = 30 inches (762 mm).

The first section, called the lead or starter, contains the helix plates. This lead section can consist of a single helix or multi-helices, typically up to four. Additional helix plates can be added, if required, with the use of helical extensions. Standard helix sizes and projected areas are shown in Table 5-4. Comprehensive tables of helix projected areas, showing both the full plate area and the area less the shaft for both square shaft and pipe shaft helical piles, is included in Section 7 of this Manual. The helix plates are usually arranged on the shaft such that their diameters stay the same size or increase as they get farther from the pilot point (tip). The practical limits on the number of helix plates per anchor/pile is usually four to five if placed in a fine-grained soils and six if placed in a coarse-grained or granular soils.

5.2.3.1 Compression Loading

The ultimate capacity of a multi-helix screw-pile with an inter-helix spacing greater than or equal to 3 ($s/B \geq 3$) is generally taken as the summation of the capacities of the individual plates:

Table 5-4 Standard Helix Sizes

LEAD SECTION AND EXTENSIONS	
DIAMETER in (cm)	**AREA ft^2 (m^2)**
6 (15)	0.185 (0.0172)
8 (20)	0.336 (0.0312)
10 (25)	0.531 (0.0493)
12 (30)	0.771 (0.0716)
14 (35)	1.049 (0.0974)
16 (40)	1.385 (0.1286)

$$Q_M = \Sigma Q_H \quad \textbf{Equation 5-20}$$

where:

Q_M = Total Capacity of a Multi-Helix Screw-Pile/Helical Anchor
Q_H = Capacity of an Individual Helix

5.2.3.2 Tension Loading

As previously noted in soft clays, especially those with high Sensitivity, it may be appropriate to reduce the undrained shear strength of the undisturbed clay for design of anchors in tension to account for some disturbance of the clay as the helical plates have passed through. This is left to the discretion of the Engineer. Most of the evidence shows that in uniform soils, the tension capacity of multi-helix anchors is the same as in compression. This means that the ultimate capacity of a multi-helix helical anchor with plate spacing of 3B or more may be taken as the summation of the capacities of the individual plates:

$$Q_M = \Sigma Q_H$$

There is some evidence that shows that in tension the unit capacity of the trailing helix plates is somewhat less than the leading helix. Engineers may wish to apply a reduction factor to account for this behavior; of about 10% for each additional helix on the helical anchor.

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5.2.4. Round Shaft Screw-Piles and Helical Anchors

Screw-piles and helical anchors are available with both square shaft and round steel pipe shafts. Square shaft is used for tension applications and also for compression applications when shaft buckling or bracing is not an issue. Pipe shaft helical piles have become increasingly popular for use in compression loading for both new construction and remediation or underpinning of existing structures. They may be either single or multi-helix. Typical round shaft pile diameters range from 2-7/8 inches (73 mm) to 12 inches (305 mm). For the most part, the design is essentially the same as with square shaft screw-piles as previously described with two simple modifications: 1) some provision is usually made to include the additional load capacity developed via skin friction by the round shaft; and 2) in tension loading, the area of the helical plate is reduced to account for the central shaft as shown in Figure 5-11b. In compression loading, the full projected area of the helix plate develops capacity since the pipe generally plugs with soil.

Typically, the length of the shaft for about one helix diameter above the helix is not included in calculating shaft resistance due to skin friction. In addition, load capacity due to friction along the pile shaft is generally mobilized only if the shaft diameter is at least 3 inches (89 mm).

5.2.4.1 Shaft Resistance in Clay φ' = 0; c' > 0

In clays, the shaft resistance developed by round shaft screw-piles and helical anchors is considered in much the same way that shaft resistance in a driven pile develops. In this traditional approach that is used for many driven piles in clays and available in most textbooks, the available "adhesion" between the shaft and the clay is obtained as a percentage of the undrained shear strength of the clay. This is the undrained or "Alpha" method in which:

$\alpha = f_S/s_u$ **Equation 5-21**

where:

α = Adhesion Factor
f_S = Unit Side Resistance
s_u = Undrained Shear Strength of the Clay

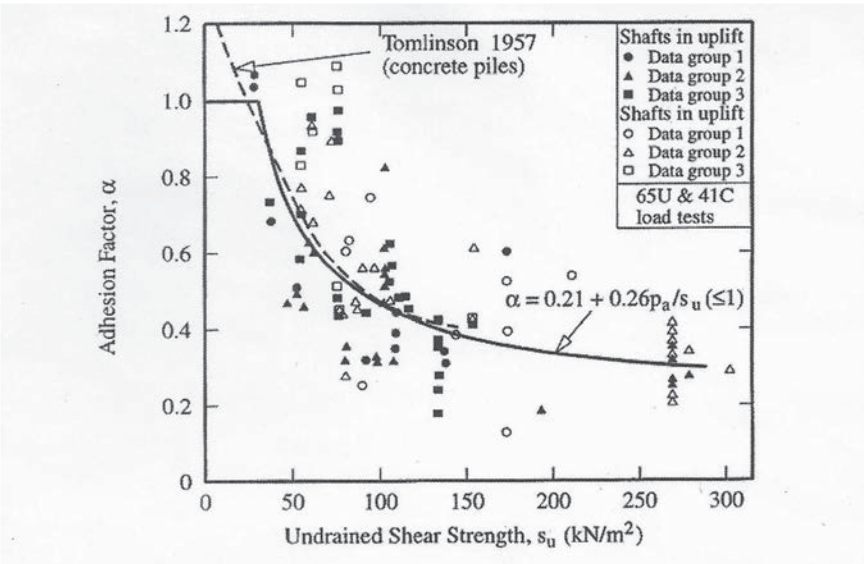

FIGURE 18.1 *Adhesion as a function of undrained shear strength*

Figure 5-8 Variation in Adhesion Factor with Undrained Shear Strength of Clays [from Canadian Foundation Manual (2006)].

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The value of α is usually obtained from any one of a number of published charts and is typically related to the absolute value of the undrained shear strength of the clay. Figures 5-8 and 5-9 give typical plots of α vs. undrained shear strength for a number of cases in which f_S has been back calculated from actual pile load tests. Generally it is sufficient to select an average value of α for a given undrained shear strength for use in design.

The total shaft resistance is then obtained from:

$Q_S = (f_S)(\pi)(d)(L)$ **Equation 5-22**

where:

Q_S = Total Shaft Resistance
d = Diameter of Central Shaft
L = Length of Round Shaft in Contact with Soil

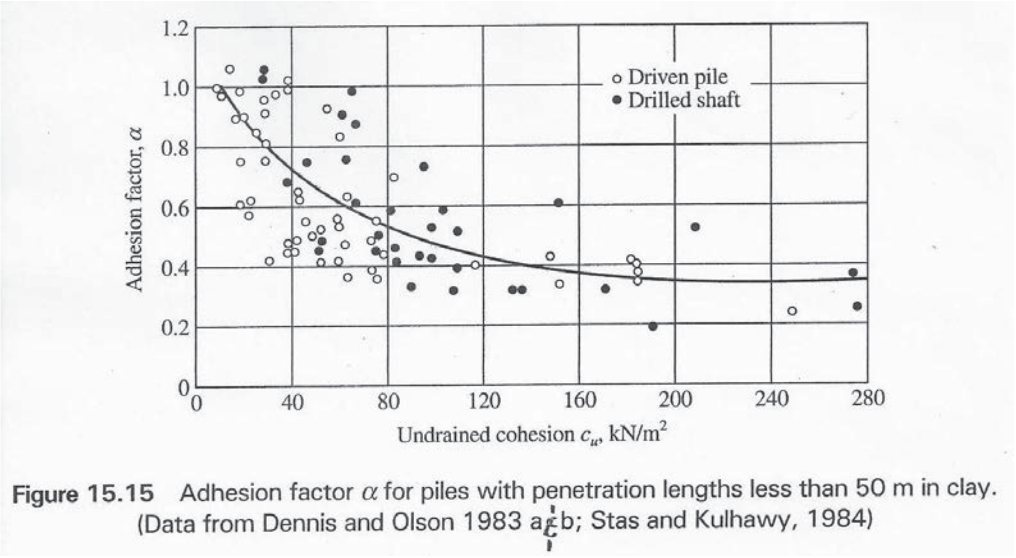


Figure 5-9 Variation in Adhesion Factor with Undrained Shear Strength of Clays (from Murthy 2003).

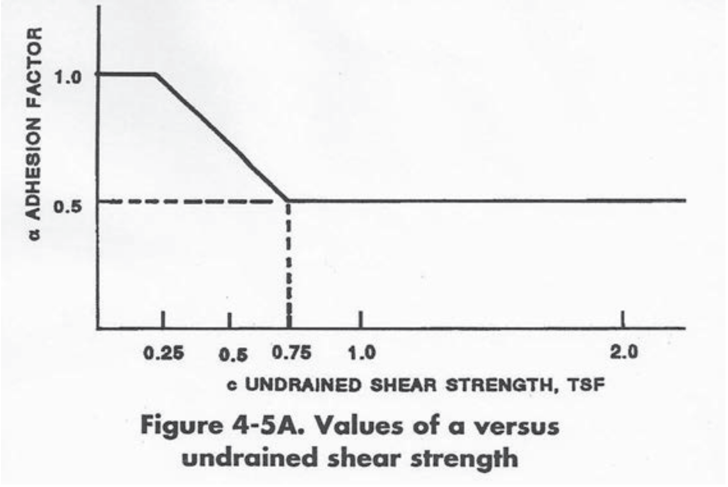
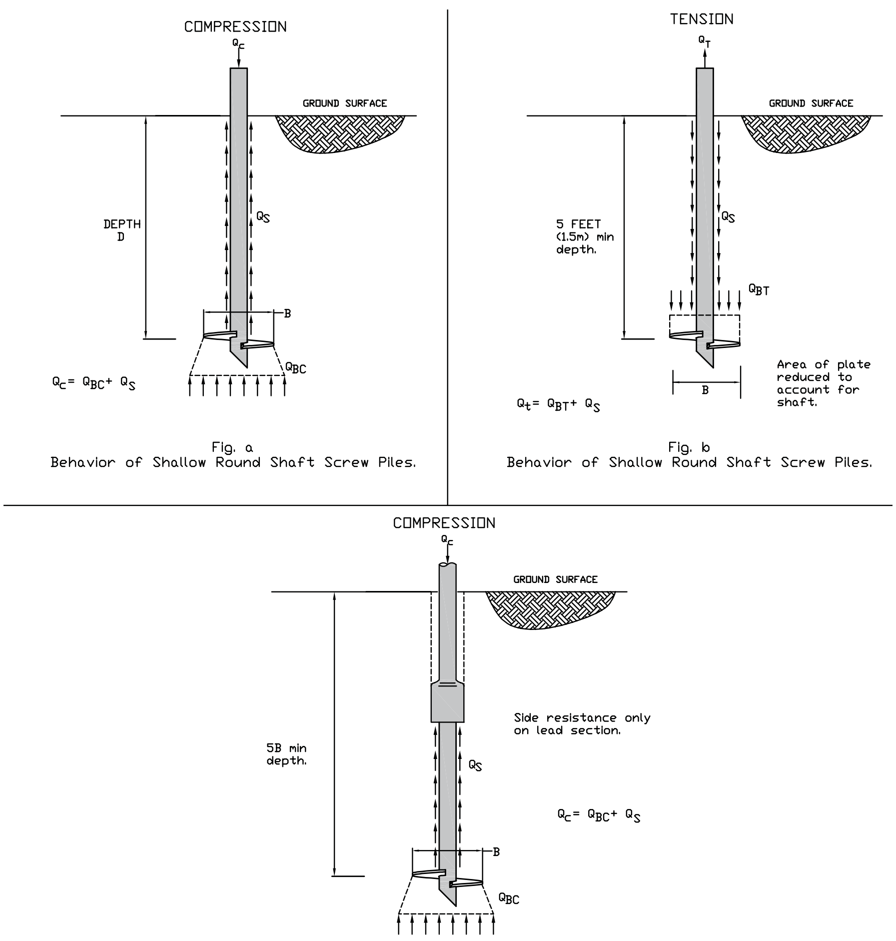


Figure 5-10 Variation in Adhesion Factor from American Petroleum Institute [from ASCE (1993b)].



Individual Bearing and Skin Friction Models for Helical Piles with Round (Pipe) Shafts
Figure 5-11

The design line given by the American Petroleum Institute (API) shown Figure 5-10 may also be used in which:

For $s_u < 500$ psf; $\alpha = 1.0$

For $s_u > 1500$ psf; $\alpha = 0.5$

For 500 psf < su < 1500 psf; α varies linearly between 1.0 and 0.5

The shaft resistance should only be calculated for that portion of the shaft length that is in full contact with the soil. This will depend on the length of the lead section, the design of the shaft couplings that connect the pile sections, and the type of soil. For example, flanged and bolted connections generally create an annulus between the shaft and the soil as the pile or anchor is installed as shown in Figure 5-11. This is because the coupling, being larger than the shaft, displaces and compacts soil. Generally, the length of the central shaft between couplings is not considered to develop shaft resistance unless the disturbed soil moves back against the shaft, or sufficient time is allowed for the soil to recover. In this situation, reduced shear strength should be used for shaft resistance capacity.

On the other hand, in the case of true flush connections between extension sections, the entire shaft may develop side resistance.

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Table 5-5 Values of Unit Side Resistance for Steel Piles in Sand (from Navy Manual DM-7)

σ'_{vo} (psf)	Friction Angle of Soil φ'				
	20	25	30	35	40
	Unit Side Resistance f_S (psf)				
500	137	175	217	263	315
1000	273	350	433	525	629
1500	410	524	650	788	944
2000	546	700	866	1050	1259
2500	683	875	1082	1313	1574
3000	819	1049	1300	1575	1888
3500	956	1244	1516	1838	2203
4000	1092	1399	1732	2101	2517

5.2.4.2 Shaft Resistance in Sand and Mixed Soils $\varphi' > 0$; $c' = 0$

The shaft resistance of steel pipe shaft piles in coarse-grained soils, such as sands and mixed soils is more complex than in clays but can still be determined using traditional deep foundation analyses. The Department of Navy Design Manual DM-7 also gives a simplified method for estimating the unit side resistance for straight shaft steel piles. The value of f_S is related to the friction angle of the soil, φ', and the effective vertical stress, σ'_{vo}, as given in Table 5-5.

5.2.5 HELICAL ANCHOR/PILE SPACING & MINIMUM DEPTH

Reasonability Check

Consideration should be given to the validity of the values obtained when determining the bearing capacity and shaft resistance of the soil. The calculated theoretical ultimate capacity is no better than the data used to obtain that value. Data from soils reports, boring logs, the water table depth, and load information may not accurately represent actual conditions where the helical pile/anchor must function. Empirical values that are used and estimates of strength parameters, etc. that must be made because of lack of data affect the calculated bearing capacity and shaft resistance value. In those situations where soil data is insufficient or not available, a helical trial probe pile can help determine such items as, location of bearing strata, pile capacity, location of soft/loose soil, and the presence of obstructions, such as, cobbles, boulders, and debris.

An important step in the process of determining the capacity of a helical pile/anchor is to conduct a reasonability check. The engineer should use the best engineering judgment to perform the reasonability check. This should be based on experience, historical test data and consulting colleagues. This is easily overlooked but must be performed by the designer or by others.

Helical Pile/Anchor Spacing

Once the capacity of the helical pile/anchor is determined, concern may turn to location of the foundation element with respect to the structure and to other helical pile/anchors. It is recommended that the center-to-center spacing between adjacent anchors/piles be no less than five times the diameter of the largest helix. The minimum spacing is three feet (0.91 m). This latter spacing should be used only when the job can be accomplished no other way and should involve special care during installation to ensure that the spacing does not decrease with depth. Minimum spacing requirements apply only to the helix bearing plate(s), i.e., the pile/anchor shaft can be battered to achieve minimum spacing. Spacing between the helical anchors/piles and other foundation elements, either existing or future, requires special consideration and is beyond the scope of this section.

Group effect, or the reduction of capacity due to close spacing, has never been accurately measured with helical piles. However, bearing capacity theory would indicate that capacity reduction due to group effect is possible, so it's considered good practice to install helical piles into dense bearing stratum when center-to center spacing is less than 4 feet (1.2 m).

Minimum Depth

As mentioned earlier, the minimum embedment depth recommended by Hubbell Power Systems, Inc. for a helical deep foundation is five helix diameters (5D), where D is the diameter of the largest helix. The 5D depth is the vertical distance from the surface to the top-most helix. Standard practice is to locate the top-most helix 6D to 8D vertical below the ground surface where practical. Minimum depth is also a function of other factors, such as seasonally frozen ground, "active" zones (depth of wetting) and depth of compressive soils. These factors are generally related to seasonal variations to soil strength parameters, but can also be related to long-term conditions, such as periods of drought or extended wet conditions. The minimum embedment depth recommended by Hubbell Power Systems, Inc. for a helical deep foundation due to seasonal variations is three diameters (3D) below the depth of soil where these seasonal variations will occur. For example, frost depths may require embedment depths that exceed the 5D minimum, depending on the project location. ICC-ES Acceptance Criteria AC358 has specified a minimum depth for helical tension anchors. AC358 states that for tension applications, as a minimum, the helical anchor must be installed such that the minimum depth from the ground surface to the uppermost helix is 12D, where D is the diameter of the largest helix. This disparity between minimum depth requirements can be reconciled by reviewing published literature on the subject, or by performing load tests.

Critical Depth

In granular soils, helical pile capacity is a function of both angle of internal friction (φ) and vertical effective overburden stress. Therefore, as a helical pile is extended deeper into soil, theoretical methods predict that the pile capacity would increase without limit as the effective vertical stress increases with increasing depth. In reality, there may be a critical depth where any further increase in depth results in only a small increase in the bearing capacity of the helical pile. Critical depth for helical piles is best determined by an experienced foundation engineer. Hubbell Power Systems, Inc. recommends critical depths of 20D to 30D be used in loose saturated soils at deep depth, where D is the diameter of the largest helix plate. The 20D to 30D length is the depth into a suitable bearing stratum, and is not necessarily measured from the ground surface.

Table 5-6 Soil Properties Required for Helical Pile/Anchor/Pile Design for Various Site Conditions

	Required Soil Properties		
Soil Property Category	**Saturated Fine-Grained**	**Coarse-Grained**	**Unsaturated Fine-Grained, Mixed Soils**
Shear Strength	s_u	φ'	c', φ'
Unit Weight	γ_{sat}	γ_{wet} or γ_{buoy}	γ_{wet}

5.3 EVALUATING SOIL PROPERTIES FOR DESIGN

The design of helical piles/anchors using the traditional soil mechanics approach described in the previous section requires evaluation of soil properties for input into the various bearing and friction capacity equations. Table 5-6 summarizes the soil properties for different site conditions for design of both single-helix and multi-helix helical piles/anchors.

Geotechnical design of helical piles/anchors requires information on the shear strength of saturated fine-grained soils, i.e., undrained shear strength, s_u, and the drained friction angle of coarse-grained soils, φ'. The best approach to evaluating these properties for design is a thorough site investigation and laboratory testing program on high quality undisturbed samples. However, this is not always possible or practical and engineers often rely on information obtained from field testing, such as the Standard Penetration Test (SPT). Whenever possible, other high quality field tests, such as Field Vane Tests (FVT), Cone Penetration Tests (CPT), Piezocone Tests (CPTU), Dilatometer Tests (DMT), Pressuremeter Tests (PMT) or Borehole Shear Tests (BST) are preferred. THERE IS NO SUBSTITUTE FOR A SITE SPECIFIC GEOTECHNICAL INVESTIGATION.

Estimating Undrained Shear Strength, s_u, in clays:

The undrained shear strength of saturated clays, silty clays and clayey silts is not a unique soil property, like Liquid Limit of clay content, but depends on the test method used for the measurement. Correlations are available for estimating undrained shear strength from the results obtained from several of the field tests noted above. The most common field results that may be available to engineers for design of helical piles/anchors are the SPT and CPT/CPTU.

s_u from SPT

A number of correlations exist for estimating both the undrained shear strength and unconfined compressive strength, q_u, of fine-grained soils from SPT results. Several of these correlations are given in Tables 5-7 and 5-8. The undrained shear strength is generally taken as one-half the unconfined compressive strength. Caution should be used when using these correlations since they have been developed for specific geologic deposits and the SPT field procedure used may not have been the same in all cases.

s_u from CPT/CPTU

The undrained shear strength may also be estimated from the tip resistance obtained from the total cone tip resistance from a CPT or the effective (net) cone tip resistance from a CPTU (e.g., Lunne et al. 1995).

Estimating s_u from the CPT total tip resistance is from a form of the bearing capacity equation as:

$$s_u = (q_c - \sigma_{vo})/N_k$$ **Equation 5-23**

where:

q_c = CPT tip resistance
σ_{vo} = total vertical stress at the cone tip = depth x total soil unit weight
N_k= empirical cone factor

The value of N_k varies somewhat with soil stiffness, plasticity, stress history and other factors, however many reported observations where s_u has been obtained from both laboratory triaxial tests and field vane tests suggest that a reasonable value of N_k for a wide range of soils is on the order of 16.

Estimating s_u from the CPTU effective tip resistance uses a modified approach since the tip resistance is corrected for pore pressure effects to give the effective tip resistance, q_t, as the undrained shear strength is obtained from:

$$s_u = (q_t - \sigma_{vo})/N_{kt}$$ **Equation 5-24**

where:

q_t = CPTU effective tip resistance
N_{kt}= empirical cone factor

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Table 5-7. Reported Correlations Between SPT N-Value and Undrained Shear Strength, s_u

Correlation to Undrained Shear Strength	Units of s_u	Soil Type	Reference
$s_u = 29N^{0.72}$	kPa	Japanese cohesive soils	Hara et al. (1974)
$s_u = 4.5N$	tsf	Insensitive Overconsolidated Clays in U.K.	Stroud (1974)
$s_u = 8N$ $N < 10$ $s_u = 7N$ $10 < N < 20$ $s_u = 6N$ $20 < N < 30$ $s_u = 5N$ $30 < N < 40$	kPa	Guabirotuba Clay	Tavares (1988)
$s_u = 1.39N + 74.2$	tsf	tropical soil	Ajayi & Balogun (1988)
$s_u = 12.5N$ $s_u = 10.5N_{60}$	kPa tsf	Sao Paulo overconsolidated clay	Decourt (1989)

Note: 1 kPa = 20.9 psf

Table 5-8. Reported Correlations Between SPT N-Value and Unconfined Compressive Strength, q_u

Correlation to Unconfined Compressive Strength	Units of q_u	Soil Type	Reference
$q_u = 12.5N$	kPa	Fine-Grained	Terzaghi & Peck (1967)
$q_u = N/8$	tsf	Clay	Golder (1961)
$q_u = 25N$ $q_u = 20N$	kPa kPa	Clay Silty Clay	Sanglerat (1972)
$q_u = 25N$ $q_u = 15N$ $q_u = 7.5N$	kPa	Highly Plastic Clay Medium Plastic Clay Low Plasticity Clay	Sowers (1979)
$q_u = 24N$	kPa	Clay	Nixon (1982)
$q_u = 62.5 (N-3.4)$	kPa		Sarac & Popovic (1982)
$q_u = 15N$	kPa	CL and CL-ML	Behpoor & Ghahramani (1989)
$q_u = 58N^{0.72}$	kPa	Fine-Grained	Kulhawy & Mayne (1990)
$q_u = 13.6\ N_{60}$ $q_u = 9.8N_{60}$ $q_u = 8.6N_{60}$ $q_u = (0.19PI + 6.2)N_{60}$	kPa	CH CL Fine-Grained Fine-Grained	Sivrikaya & Togrol (2002)

The value of N_{kt} also has been shown to vary for different soils but a reasonable conservative value for massive clays is on the order of 12. For very stiff, fissured clays, the value of N_{kt} may be as high as 30.

Other methods are available for estimating undrained shear strength from the pore pressure measurements from a CPTU or by first estimating the stress history from CPT/CPTU results and then converting to undrained shear strength, e.g., NCHRP (2007); Schnaid (2009), both of which are viable approaches.

Estimating Shear Strength of Fine-Grained Soil – Other Methods

Vane Shear Test: Shear strength of fine-grained soils may be measured both in the field and in the laboratory. One of the most versatile devices for investigating undrained shear strength and sensitivity of soft clays is the vane shear test. It generally consists of a four-bladed rectangular vane fastened to the bottom of a vertical rod. The blades are pressed their full depth into the clay surface and then rotated at a constant rate by a crank handle. The torque required to rotate the vane is measured. The shear resistance of the soil can be computed from the torque and dimensions of the vane.

One such type of the portable vane shear test is the Torvane. It is a convenient hand-held device useful for investigating the strength of clays in the walls of test pits in the field or for rapid scanning of the strength of Shelby tubes or split spoon samples. A calibrated spring allows undrained shear strength (cohesion) to be read directly from the indicator.

Pocket Penetrometer Test: Another device used to estimate undrained shear strength in the laboratory or the field is the Pocket Penetrometer. As with the vane shear test, the pocket penetrometer is commonly used on Shelby tube and split spoon samples, and freshly cut test pits to evaluate the consistency and approximate unconfined compressive strength (q_u) of clay soils. The penetrometer's plunger is pushed into the soil ¼" and a reading taken on the sliding scale on the side. The scale is a direct reading of shear strength. Pocket Penetrometer values should be used with caution. It is not recommended for use in sands or gravel soils.

Unconfined Compression Test: The unconfined compression (UC) test is used to determine the consistency of saturated clays and other cohesive soils. A cylindrical specimen is set up between end plates. A vertical load is applied incrementally at such a rate as to produce a vertical strain of about 1 to 2% per minute – which is rapid enough to prevent a volume change in the sample due to drainage. The unconfined compressive strength (q_u) is considered to be equal to the load at which failure occurs divided by the cross-sectional area of the sample at the time of failure. In clay soils where undrained conditions are expected to be the lower design limit (i.e. the minimum Factor of Safety), the undrained shear strength (i.e., cohesion) governs the behavior of the clay. This undrained shear strength is approximately equal to ½ the unconfined compressive strength of undisturbed samples (see Laboratory Testing of Recovered Soil Samples in Section 2 of this Technical Manual).

The consistency of clays and other cohesive soils is usually described as soft, medium, stiff, or hard. Tables 5-9 and 5-10 can be found in various textbooks and are reproduced from Bowles, 1988. Values of consistency, overconsolidation ratio (OCR), and undrained shear strength (cohesion) empirically correlated to SPT N-values per ASTM D 1586 are given in Table 5-9. It should be noted that consistency correlations can be misleading because of the many variables inherent in the sampling method and the soil deposits sampled. As such, Table 5-9 should be used as a guide.

The relative density of sands, gravels, and other granular soils is usually described as very loose, loose, medium dense, dense, very dense, or extremely dense. The standard penetration test is a good measure of granular soil density. Empirical values for relative density, friction angle and unit weight as correlated to SPT "N" values per ASTM D 1586 are given in Table 5-10. It should be noted that SPT values can be amplified in gravel because a 1"+ gravel particle may get lodged in the opening of the sampler. This can be checked by noting the length of sample recovery on the soil boring log (see Table 2-6). A short recovery in gravelly soils may indicate a plugged sampler. A short or "low" recovery may also be indicated by loose sand that falls out of the bottom of the sampler during removal from the borehole.

Estimating Friction Angle, φ′, in sands

Results from both the SPT and CPT may be used to estimate the drained friction angle of sands and other coarse-grained soils. Generally, most site investigations involving coarse-grained soils will include the use of either the Standard Penetration Test (SPT) or the Cone Penetrometer (CPT).

φ′ from SPT

Several correlations have been proposed to estimate the drained friction angle in sands from SPT results. A summary of several of the more popular correlations are given in Table 5-11. The correlation of Hatanaka & Uchida (1996) is shown in Figure 5-12, taken from FHWA Reference Manual on Subsurface Investigations (2002).

Table 5-9. Terms to Describe Consistency of Saturated Cohesive Soils

Consistency Term	Stress History	SPT N_{60}-Values	Undrained Shear Strength skf (kPa)	Comments
Very Soft	Normally Consolidated OCR = 1	0 - 2	<0.25 (12)	Runs through fingers.
Soft	Normally Consolidated OCR ≅ 1 – 1.2	3 - 5	0.38 (18.2) to 0.63 (30.2)	Squeezes easily in fingers.
Medium	Normally Consolidated OCR = 1 to 2	6 - 9	0.75 (36) to 1.13 (54.1)	Can be formed into a ball.
Stiff	Normally Consolidated to OCR of 2-3.	10 - 16	1.25 (59.9) to 2 (95.8)	Hard to deform by hand squeezing.
Very Stiff	Overconsolidated OCR = 4 – 8	17 - 30	2.13 (102) to 3.75 (179.6)	Very hard to deform by hand.
Hard	Highly Overconsolidated OCR > 8	>30	>3.75 (179.6)	Nearly impossible to deform by hand.

φ' from CPT/CPTU

A similar approach may be used to estimate the friction angle of sands from the CPT/CPTU tip resistance based on a modified bearing capacity theory. Robertson and Campanella (1983) summarized a number of available calibration chamber tests on five sands and suggested a simple correlation between the normalized CPT tip resistance and a cone bearing capacity factor, N_q as:

$$N_q = (q_c/\sigma_{v0}') = 0.194\exp(7.63\tan\varphi') \quad \textbf{Equation 5-26}$$

where:

σ_{v0}' = vertical effective (corrected for pore water pressure) stress at cone tip

This relationship is shown in Figure 5-14.

The friction angle may also be estimated from the effective tip resistance from the CPTU. Early calibration chamber data suggested a simple empirical correlation as:

$$\varphi' = \arctan[0.1 + 0.38 \log (q_t/\sigma'_{vo})] \quad \textbf{Equation 5-27}$$

Equation 5-27 is shown in Figure 5-16.

Table 5-10. Empirical Values for Dr, Friction Angle and Unit Weight vs SPT (Assuming a 20 ft (6 m) depth of overburden and 70% rod efficiency on hammer)

Description		Very Loose	Loose	Medium	Dense	Very Dense
Relative Density (D_r) (%)		0	15	35	65	85
SPT (N_{70})	Fine	1-2	3-6	7-15	16-30	?
	Medium	2-3	4-6	8-20	21-40	40+
	Coarse	3-6	5-9	10-25	26-45	45+
Friction Angle (ϕ)	Fine	26-28	28-30	30-33	33-38	38+
	Medium	27-29	29-32	32-36	36-42	50+
	Coarse	28-30	30-34	34-40	40-50	50+
Total Unit Weight (g_{wet}) (PCF)		70-100	90-115	110-130	110-140	130-150

Additional test results from 24 different sands were compiled by Kulhawy and Mayne (1990) who proposed the following expression:

$\varphi' = 17.70 + 11.0 \log (q_{t1})$ **Equation 5-28**

where:

$(q_{t1}) = (q_t/\sigma_{atm})/(\sigma'_{vo}/\sigma_{atm})^{0.5}$

σ_{atm} = atmospheric pressure (1 atm = 1 bar = 100 kPa = 1tsf = 14.7 psi)

Table 5-11. Reported Correlations between SPT N-Value and φ' for Coarse-Grained Soils

Correlation	Reference
$\varphi' = (0.3N)^{0.5} + 27^0$	Peck et al. (1953)
$\varphi' = (10N)/35 + 27^0$	Meyerhof (1956)
$\varphi' = (20N)^{0.5} + 15^0$	Kishida (1967)
$\varphi' = (N/\sigma'_{vo})^{0.5} + 26.9^0$ (σ'_{vo} in MN/m^2)	Parry (1977)
$\varphi' = (15N)^{0.5} + 15^0$	Shioi & Fukui (1982)
$\varphi' = (15.4(N_1)_{60})^{0.5} + 20^0$	Hatanaka & Uchida (1996)

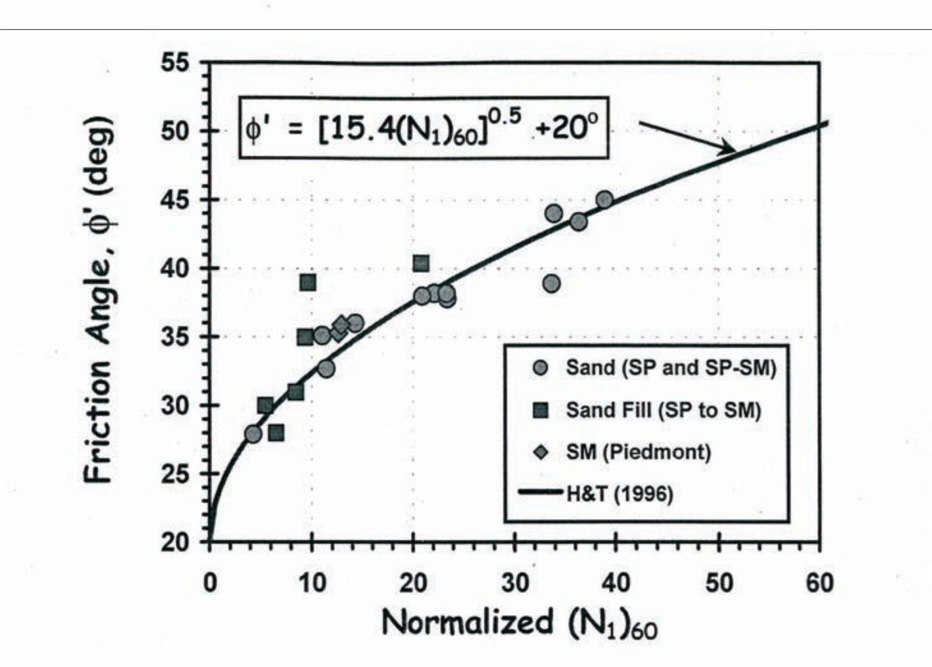
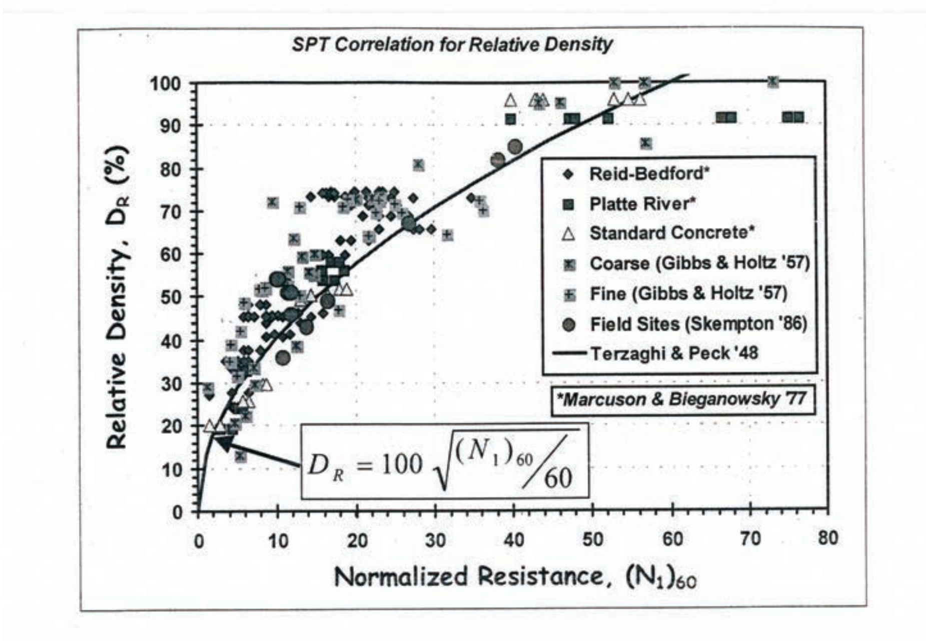


Figure 5-12 Peak Friction Angle of Sands from SPT Resistance - Correlation of Hatanaka & Uchida (1996) from FHWA Reference Manual on Subsurface Investigations (2002)



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Direct Estimate of Unit Shaft Resistance, f_s, of Steel Round Shaft Piles and Grouted Helical Micropiles

Suggestions for estimating the unit side resistance, f_s, of deep foundations in a variety of soils have been presented. This approach is convenient for helical piles/anchors and reduces assumptions in first estimating shear strength and then estimating other factors to obtain f_s. Poulos (1989) summarized a number of reported correlations between pile unit side resistance and SPT N-value and suggested that most of these correlations could be expressed using the general equation:

$$f_s = \beta + \alpha N \qquad \textbf{Equation 5-29}$$

Lutenegger (2011) presented a summary of more-or-less "global" reported correlations between SPT N-values and unit side resistance friction for both driven and bored piles in a number of different soil materials and shown in Table 5-12.

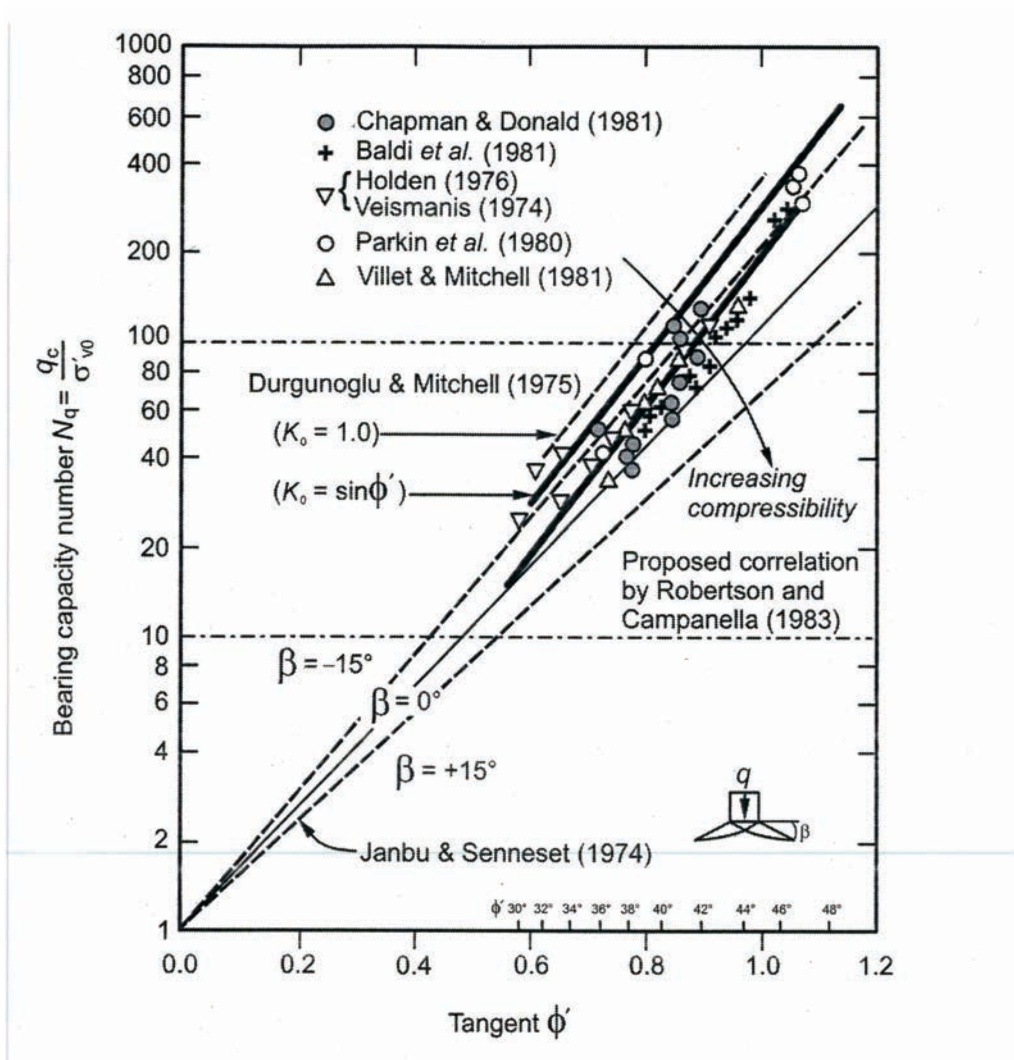

Figure 5-14. Relationship between Bearing Capacity Number and Friction Angle from Normalized CPT Tip Resistance – from Robertson and Campanella (1983)

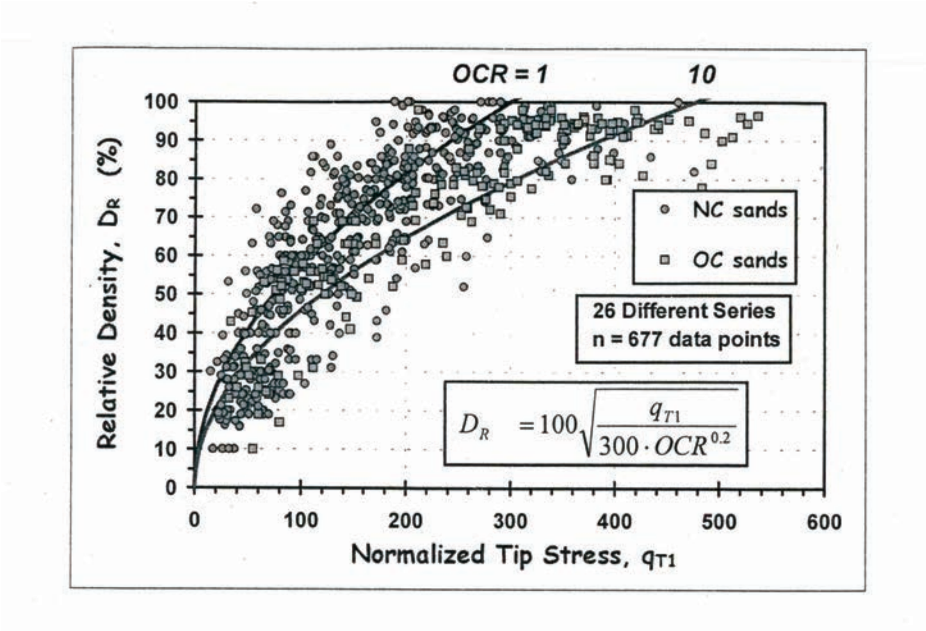

Figure 5-15. Relationship Between Relative Density for Normally Consolidated (NC) and Over Consolidated (OC) Sands from CPT Data.

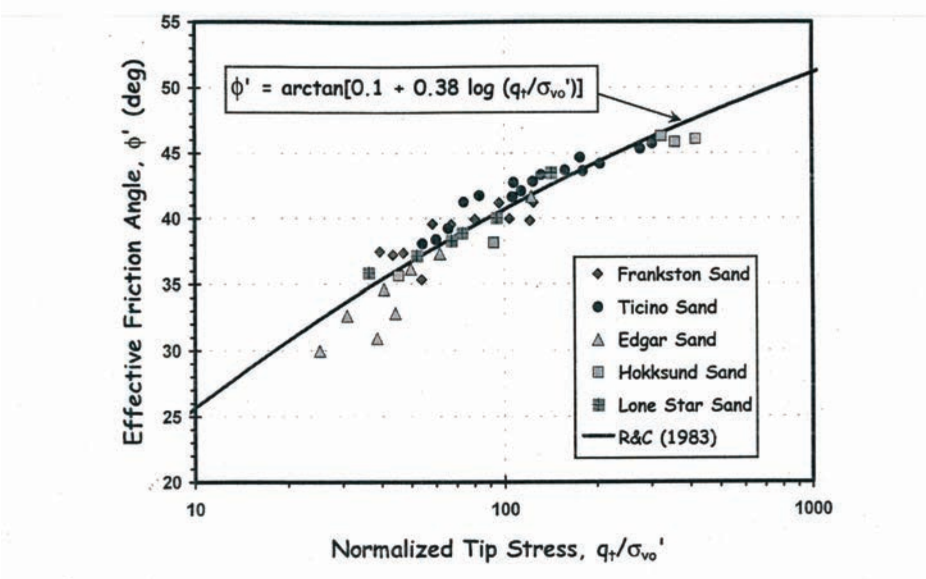

Figure 5-16. Relationship Between Friction Angle and the Effective Tip Resistance from CPTU Data

$(N_1)_{60} = N_{60}/(\sigma'_{vo})^{0.5}$ σ'_{vo} = effective overburden stress in tsf

Engineers might ask "Why should the SPT N-value correlate to pile side resistance?" Other than being purely coincidental, there must be a rational and logical explanation for such observations. The range in reported values of α given in Table 5-12 is quite large and the results might seem of limited use. Nonetheless, we can make some general observations and summarize these observations: 1) For most of these correlations, the value of β is very low and for practical purposes may be reasonably taken as zero with little effect on the correlation, which simplifies Eq. 5-29 to:

$$fs = \alpha N$$ **Equation 5-30**

2) The value of α ranges from 0.3 to 12.5; 3) The observations presented in Table 5-12 generally suggest higher values of α for fine-grained soils as compared to coarse-grained soils; and 4) Values of α are generally higher for driven piles as compared to bored piles.

The values of α vary considerably for a number of obvious reasons, deriving from both the pile data as well as the SPT data. In regard to the pile data: 1) The data represent a wide range of pile types, i.e., different geometry, such as open and closed end pipe, H-Piles and construction practices; such as dry bored vs. wet bored as well as pile size, pile plugging, L/d, and other factors; 2) Different methods may have been used to interpret the ultimate capacity and to isolate the side resistance from end bearing; 3) The unit side resistance from pile tests is typically averaged over the length of the pile except in the case of well instrumented piles. Regarding the SPT data:1) The results most likely represent a wide range in field practice including a wide range in energy or hammer efficiency; 2) It is likely that other variations in field practice or equipment such as spoon geometry are not consistent among the various studies and may affect results. Engineers should use the correlations in Table 5-12 with caution.

In fact, Equation 5-30 is similar to Equation 5-21, suggesting a correlation between SPT N-values and undrained shear strength (s_u) in fine-grained soils.

5.4 FACTOR of SAFETY

The equations discussed above are used to obtain the ultimate capacity of a helical anchor/pile. For working, or allowable stress design (ASD), an appropriate Factor of Safety must be applied to reduce the ultimate capacity to an acceptable design (or working) capacity. The designer determines the Factor of Safety to be used. In general, a minimum Factor of Safety of 2 is recommended. For tieback applications, the Factor of Safety typically ranges between 1.25 and 2.

Design or working loads are sometimes referred to as unfactored loads and do not include any Factor of Safety. They may arise from dead loads, live loads, snow loads and/or earthquake loads for bearing (compression) loading conditions; from dead loads, live loads, snow loads and/or wind loads for anchor loading conditions; and earth pressure, water pressure and surcharge loads (from buildings, etc.) for helical tieback or SOIL SCREW® earth retention conditions.

Ultimate loads, sometimes referred to as fully factored loads, already fully incorporate a Factor of Safety for the loading conditions described above. Hubbell Power Systems, Inc. recommends a minimum Factor of Safety of 2.0 for permanent loading conditions and 1.5 for temporary loading conditions. This Factor of Safety is applied

Table 5-12. Reported Correlations between SPT N-Value and Pile Side Resistance (from Lutenegger 2011)

Pile Type	Soil	β	α	Reference
driven displacement	granular	0	2.0	Meyerhof (1976)
	miscellaneous soils .(f_s < 170 kPa)	10	3.3	Decourt (1982)
	cohesive	0	10	Shioi & Fukui (1982)
	cohesive cohesionless	0 0	3 1.8	Bazaraa & Kurkur (1986)
	sandy clayey	29 34	2.0 4.0	Kanai & Yubuuchi (1989)
	misc	0	1.9	Robert (1997)
bored	granular	0	1.0	Meyerhof (1976)
	granular	55	5.8	Fujita et al. (1977)
	cohesionless	0	3.3	Wright & Reese (1979)
	cohesive (f_s < 170 kPa)	10	3.3	Decourt (1982)
	cohesive	0	5.0	Shioi & Fukui (1982)
	cohesive cohesionless	0 0	1.8 0.6	Bazaraa & Kurkur (1986)
	residual soil & weathered rock	0	2.0	Broms et al. (1988)
	clay sand	0 0	1.3 0.3	Koike et al. (1988)
	sandy soil cohesive	35 24	3.9 4.9	Kanai & Yubuuchi (1989)
	residual soil	0	4.5	Winter et al. (1989)
	gravel sand silt clay	0 0 0 0	6.0 4.0 2.5 1.0	Hirayama (1990)
	residual soils	0	2.0	Chang & Broms (1991)
	clayey soil sandy soil	0 0	10.0 3.0	Matsui (1993)
	misc.	17.3 18.2	1.18 0.65	Vrymoed (1994)
	misc.	0	1.9	Robert (1997)
	sand	0	5.05	Kuwabara & Tanaka (1998)
	weathered rock	0	4	Wada (2003)
cast-in-place	cohesionless cohesive	0 0	5.0 10.0	Shoi & Fukui (1982)
	cohesionless (f_s < 200 kPa) cohesive (f_s < 150 kPa)	30 0	2.0 5.0	Yamashita et al.(1987)

Note: $f_s = \beta + \alpha N$ (f_s in units of kPa)

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to the design or working loads as defined above to achieve the ultimate load requirement. National and local building code regulations may require more stringent Factors of Safety on certain projects.

Most current structural design standards in Canada use a Limit States Design (LSD) approach for the structural design of helical piles/anchors rather than working or allowable stress design (WSD). All specified loads (dead, live, snow, wind, seismic, etc.) are factored in accordance with appropriate load factors and load combinations should be considered. In addition, the geotechnical resistance of the helical pile/anchor must be factored. Geotechnical resistance factors for helical piles/anchors are not yet clearly defined. Therefore, a rational approach should be taken by the designer and resistance factors should be considered that are suitable to specific requirements.

5.5 HeliCAP® HELICAL CAPACITY DESIGN SOFTWARE

Hubbell Power Systems, Inc. engineers developed HeliCAP® design software to determine the bearing capacity of helical piles and anchors in soil. Since then, it has been revised several times to provide additional features such as side resistance for steel pipe piles and grouted shaft helical piles. HeliCAP® software is available to engineers and designers upon request. The software uses the same theory of general bearing capacity as presented in Section 5.2 for deep foundations (minimum depth ≥ 5D). A key feature of HeliCAP is it's designed to work with the information commonly available from soils reports. In North America, soil investigation usually includes a soil boring as described in Section 2 of this Technical Design Manual. The most common information available from the soils boring is the soil profile, groundwater location, and SPT blow count data per ASTM D-1586. As such, HeliCAP® includes blow count correlations for shear strength, angle of internal friction, and unit weight. These correlations are generally accepted as reasonable approximations given the available blow count data.

The following equations, factors, empirical values, etc., presented in this section are the algorithms used in the HeliCAP® v2.0 Helical Capacity Design Software. This program makes the selection of a helical anchor/pile much quicker than making hand calculations. It allows calculations to be made quickly while varying the different parameters to arrive at the most appropriate solution. As with any calculations, the results from this program are no better than the input data used to generate them.

The program will assist in determining an appropriate helical lead configuration and overall anchor/pile length. It also provides an estimate of the installation torque. The helical lead configuration can vary by the number and sizes of helix plates required to develop adequate capacity. Helical anchor/pile length may vary due to the combined effects of the lead configuration and soil strength. Generally speaking, the shorter the pile length for a given load, the better the performance will be in regard to deflection under load.

HeliCAP® BEARING CAPACITY METHODOLOGY

As detailed earlier in this Section, the Individual Plate Bearing Method states the capacity of a single or multi-helix anchor/pile is determined by summing the bearing capacity of the individual helix plate elements specific to a given pile. Thus:

$$Q_t = \Sigma Q_h$$

where:

Q_t = Total ultimate multi-helix anchor/pile capacity
Q_h = Individual helix capacity

HeliCAP determines the ultimate bearing capacity of an individual helix as per the following equation. An upper limit for this capacity is based on helix strength that can be obtained from the manufacturer. See Section 7 of this Technical Design Manual for the mechanical strengths of helix plates.

$$Q_h = A_h\,(cN_c + q'N_q) \leq Qs \qquad \textbf{Equation 5-31}$$

where:

A_h = Projected helix area
Q_s = Capacity upper limit, determined by the helix mechanical strength

Sands φ' > 0; c' = 0

HeliCAP® determines the ultimate bearing capacity in a non-cohesive sand or gravel soil with Equation 5-32 in which the fine-grain (clay) term has been eliminated.

The bearing capacity factor N_q is dependent on the angle of internal friction (φ') of the non-cohesive sand or gravel soil. When a value is provided for the friction angle, HeliCAP uses Figure 5-7 (Nq vs φ') and Equation 5-19 to determine the value for N_q. When the angle of internal friction is not known, HeliCAP estimates it (and Nq) by using blow counts obtained from the Standard Penetration Test per ASTM D 1586. Equation 5-33 allows an estimate of the angle of internal friction from SPT blow count data. This equation is based on empirical data given by Bowles (1968) and its results should be used with caution. The graph in Figure 5-7 allows the determination of N_q for a specific angle of internal friction when measured in degrees. This curve was adapted from work by Meyerhof (1976). Equation 5-19 was written for the curve shown in Figure 5-7, which is Myerhof's N_q values divided by 2 for long term applications. **Note the correlated φ' and Nq values determined by HeliCAP® can be overridden. This is encouraged when more reliable soil data are available.**

$$Q_h = A_h q' N_q = Ah\,\gamma' D N_q \qquad \textbf{Equation 5-32}$$

where:

A_h = Projected helix area
D = Vertical depth to helix plate
N_q = Bearing capacity factor for non-cohesive component of soil
γ' = Effective unit weight of the soil

$$\varphi' = 0.28\,N + 27.4 \qquad \textbf{Equation 5-33}$$

where:

φ' = Angle of internal friction
N = Blow count per ASTM D 1586 Standard Penetration Test

Fine-Grain Cohesive Soil, φ' = 0; c' > 0

HeliCAP® determines the ultimate bearing capacity in a cohesive or fine-grained soil with Equation 5-17 with the overburden term not used. The Nc factor is 9, provided the installation depth below grade is greater than five times the diameter of the top most helix.

$$Qh = A_h c N_c = A_H[(9)(s_u)] \qquad \textbf{Equation 5-34}$$

where:

A_h = Projected helix area
c = "cohesion"; for φ' = 0; c = undrained shear strength = s_u
N_c = Bearing Capacity Factor for Deep Failure = 9 (minimum depth ≥ 5D)

In the event that cohesion or undrained shear strength values are not available, HeliCAP® uses the following

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equation to obtain estimated undrained shear strength values when blow counts from ASTM D 1586 Standard Penetration Tests are available. This equation is based on empirical values and is offered only as a guide when undrained shear strength values are otherwise not available. It is suggested that results be used with caution. **(NOTE: The correlated undrained shear strength values determined by HeliCAP® can be overridden. This is encouraged when more reliable soil data are available.)**

$c\ (\mathrm{ksf}) = N / 8 \text{ or } = 0.125(N)$ **Equation 5-35**

$c\ (\mathrm{kPa}) = 6N$

where:

c = "cohesion"; for $\varphi' = 0$; c = undrained shear strength = s_u
N = Blow count value per ASTM D 1586 Standard Penetration Test

Unit Weight Correlation

In the event unit weight values are not available, HeliCAP® uses the following equations to obtain estimated unit weight values when blow counts from ASTM D 1586 Standard Penetration Tests are available.

Clay (Fine-Grain) Soils: **Equation 5-36**
$N > 0\ \&\ N \le 19$: $\gamma = 80 + (2N)\ (\mathrm{lb/ft^3})$
$N \ge 20\ \&\ N \le 40$ $\gamma = 120\ (\mathrm{lb/ft^3})$

$N \ge 41\ \&\ N < 50$ $\gamma = 120 + 2(N-40)\ (\mathrm{lb/ft^3})$ **Equation 5-37**
$N \ge 50$ $\gamma = 140\ (\mathrm{lb/ft^3})$

Sand (Coarse-Grain) Soils:
$N = 0$ $\gamma = 65\ (\mathrm{lb/ft^3})$

$N > 0\ \&\ N \le 7$ $\gamma = 60 + 5N\ (\mathrm{lb/ft^3})$ **Equation 5-38**
$N \ge 8\ \&\ N \le 10$ $\gamma = 100\ (\mathrm{lb/ft^3})$

$N \ge 11\ \&\ N < 50$ $\gamma = 90 + N\ (\mathrm{lb/ft^3})$ **Equation 5-39**
$N \ge 50$ $\gamma = 140\ (\mathrm{lb/ft^3})$

These correlations were originally determined from Tables 3-2 and 3-3 in Bowles first edition of Foundation Analysis and Design. These relationships provide an approximation of the total unit weight. They have been modified slightly from how they were originally presented as experience has suggested. **(NOTE: The correlated total unit weight values determined by HeliCAP® can be overridden. This is encouraged when more reliable soil data are available.)**

Mixed Soils φ' > 0; c' > 0

The determination of the bearing capacity of a mixed soil, one that exhibits both cohesion and friction properties, is accomplished by use of Equation 5-31. This is fairly uncomplicated when accurate values are available for both the cohesion (undrained shear strength) and friction terms (φ' & γ') of the equation. It is not possible to use ASTM D 1586 Blow Count correlations to determine all soil strength variables in the bearing capacity equation. Therefore, unless the designer is quite familiar with the project soil conditions, it is recommended that another approach be taken when accurate values are not available for both terms of the equation.

One suggestion is to first consider the soil as fine-grained (cohesive) only and determine capacity. Then consider the same soil as coarse-grained (cohesionless) only and determine capacity. Finally, take the lower of the two results and use that as the soil bearing capacity and apply appropriate Factors of Safety, etc.

HeliCAP® SHAFT RESISTANCE METHODOLOGY

As discussed earlier in this section, the shaft resistance developed by pipe shaft or grouted shaft screw-piles is considered in much the same way that shaft resistance in a driven pile develops. HeliCAP® uses this traditional approach that is available in most foundation design textbooks.

The general equation is:

$$Qf = \Sigma[\pi(D)fs(\Delta Lf)] \qquad \textbf{Equation 5-40}$$

where:

D = Diameter of steel or concrete pile column
f_s = Sum of friction and adhesion between soil and pile
ΔL_f = incremental pile length over which πD and fs are taken as constant

HeliCAP® uses two empirical methods to calculate shaft resistance - the Gouvenot Method and the US Department of Navy Method. The Gouvenot Method is named after the French researcher; who conducted tests on a variety of grouted shaft micropiles including gravity fed grout columns. HeliCAP® uses the Gouvenot method to calculate shaft resistance for grouted columns only (HELICAL PULLDOWN® Micropiles). The US Navy method uses the Dept. of Navy Design Manual 7, Soil Mechanics, Foundations and Earth Structures (1974). HeliCAP® uses the Navy method to calculate shaft resistance for both grouted columns and straight steel pipe shafts.

- Gouvenot reported a range of values for skin friction of concrete/grout columns based on a number of field load tests. The soil conditions are divided into three categories based on friction angle (ϕ) and cohesion (c). The equations used to calculate f_s are:

  Type I: Sands and gravels with 35° < φ < 45° and c' = 0:

  $$f_s = \sigma_o \tan \varphi \qquad \textbf{Equation 5-41}$$

  where: σ_o = Mean normal stress for the grout column

  Type II: Mixed soils; fine loose silty sands with 20° < φ < 30° and sandy clays with 205 psf < C < 1024 psf (9.8 kPa < c < 49 kPa)

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$f_s = \sigma_o(\sin \varphi) + c(\cos \varphi)$ **Equation 5-42**

Type III: Clays with 1024 psf < c < 4096 psf (49 kPa < c < 196 kPa)

$f_s = C$ **Equation 5-43**

where: 1024 psf < c < 2048 pfs (49 kPa < c < 98 kPa)
and:

f_s = 2048 psf (98 kPa) **Equation 5-44**

where: 2048 psf < c < 4096 psf (98 kPa < c < 196 kPa)

In HeliCAP® this analysis assumes a uniform shaft diameter for each soil layer and, if required, the friction capacity of the pile near the surface can be omitted.

- **Department of the Navy Design Manual 7 Method:**

For cohesive soils (α Method):

$Q_f = \Sigma[\pi(D)C_a(\Delta L_f)]$ **Equation 5-45**

where: C_a = Adhesion factor (See Table 5-13)

For cohesionless soils (α Method):

$Q_f = \Sigma[\pi D(qK\tan\phi)\Delta L_f]$ **Equation 5-46**

where: q = Effective vertical stress on element ΔL_f

K = Coefficient of lateral earth pressure ranging from K_o to about 1.75 depending on volume displacement, initial soil density, etc. Values close to K_o are generally recommended because of long-term soil creep effects. As a default, use K_o = 1.
φ = Effective friction angle between soil and plate material

$Q_f = \Sigma[\pi D(S)\Delta L_f]$ **Equation 5-47**

where: S = Average friction resistance on pile surface area = $P_o \tan \varphi$ (See Tables 5-5 & 5-14)
P_o = Average overburden pressure

For straight steel pipe shaft piles in sand, HeliCAP® uses Table 5-5 to calculate shaft resistance in sand layers using the Alternate Navy Method.

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Tables 5-13, 5-14 and 5-5 are derived from graphs in the Department of the Navy Design Manual 7, Soil Mechanics, Foundations and Earth Structures (1974). Later editions of this manual limit the depth at which the average overburden pressure is assumed to increase. The following is an excerpt from the manual regarding this limiting depth:

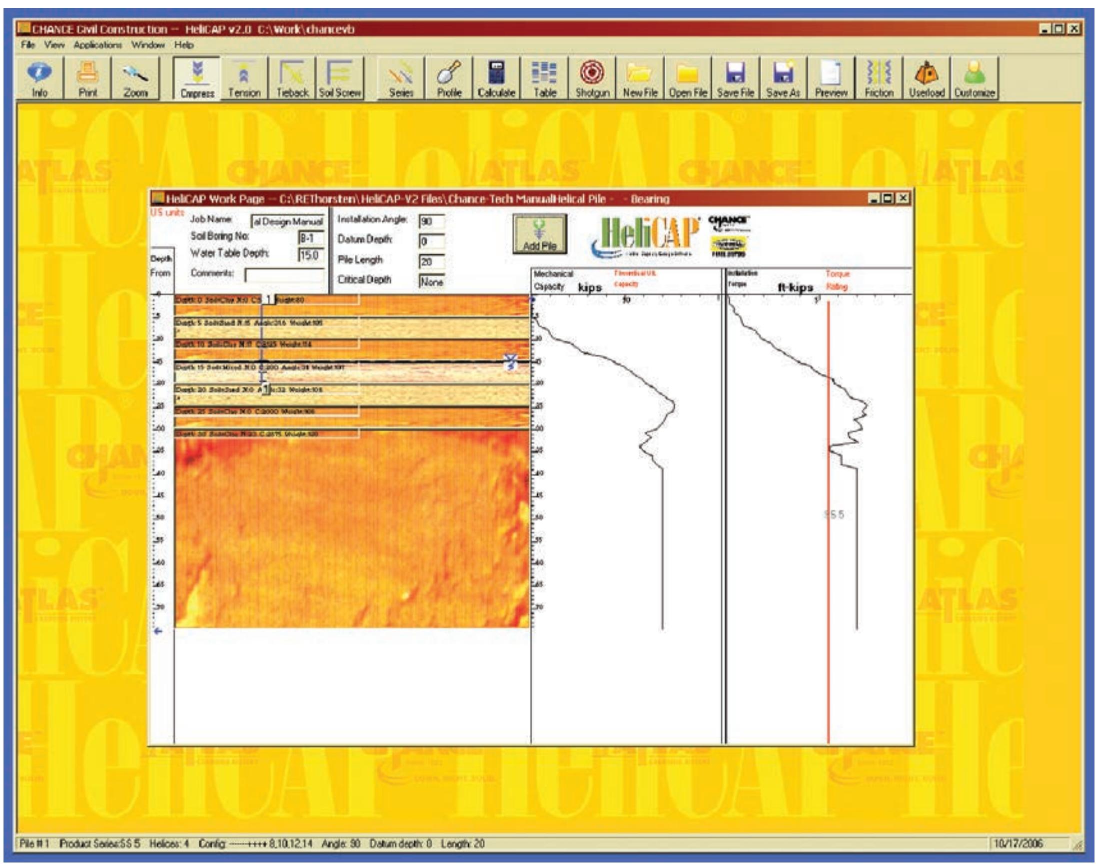
"Experimental and field evidence indicate that bearing pressure and skin friction increase with vertical effective stress (P_o) up to a limiting depth of embedment, depending on the relative density of the granular soil and position of the water table. Beyond this limiting depth (10B± to 40B±) there is very little increase in end bearing, and increase in side friction is directly proportional to the surface area of the pile. Therefore, if D is greater than 20B, limit P_o at the pile tip to that value corresponding to D = 20B" where D = depth of the pile embedment over which side friction is considered and B = diameter of the pile.

Design Example 8-5 in Section 8 illustrates the use of the Navy Design Manual 7 method to calculate the friction capacity of a CHANCE HELICAL PULLDOWN® Micropile.

HeliCAP® v2.0 Helical Capacity Design Software calculates ultimate capacity and must have an appropriate Factor of Safety applied to the results. The program has additional features that allow it to be used for other applications, but it is beyond the scope of this manual to present all facets of the program. For additional assistance, refer to the Help screen or contact Hubbell Power Systems, Inc. application engineers.

The following screen is from HeliCAP® v2.0 Helical Capacity Design Software. It shows a typical workpage with the soil profile on the left and helical pile capacity on the right.

Design Examples 8-3 through 8-12 in Section 8 illustrate the use of the standard bearing equation to determine the bearing capacities of helical piles/anchors. These design examples are taken from actual projects involving residential and commercial new construction, boardwalks, tiebacks, telecommunication towers, pipeline buoyancy control, etc.



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5.6 APPLICATION GUIDELINES for CHANCE® HELICAL PILES/ANCHORS

- The uppermost helix should be installed at least three diameters below the depth of seasonal variation in soil properties. Therefore, it is important to check the frost depth or "mud" line at the project site. Seasonal variation in soil properties may require the minimum vertical depth to exceed five helix diameters. The influence of the structure's existing foundation (if any) on the helical pile/anchor should also be considered. Hubbell Power Systems, Inc. recommends helical piles/anchors be located at least five diameters below or away from existing foundation elements.
- The uppermost helix should be installed at least three helix diameters into competent load-bearing soil. It is best if all helix plates are installed into the same soil stratum.
- For a given shaft length, use fewer longer extensions rather than many shorter extensions. This will result in fewer connections and better load/deflection response.
- Check economic feasibility if more than one combination of helical pile/anchors helix configuration and overall length can be used.

Table 5-13. Recommended Adhesion Values in Clay *

PILE TYPE	SOIL CONSISTENCY	COHESION, c (psf)	ADHESION, Ca (psf)
Concrete	Very Soft	0 – 250	0 – 250
	Soft	250 – 500	250 – 480
	Medium Stiff	500 – 1000	480 – 750
	Stiff	1000 – 2000	750 – 950
	Very Stiff	2000 – 4000	950 – 1300
Steel	Very Soft	0 – 250	0 – 250
	Soft	250 – 500	250 – 460
	Medium Stiff	500 – 1000	460 – 700
	Stiff	1000 – 2000	700 – 720
	Very Stiff	2000 – 4000	720 - 750
* From Department of the Navy Design Manual 7, Soil Mechanics, Foundations and Earth Structures (1974).			

Table 5-14. Straight Concrete Piles in Sand

P_o (psf)	Effective Angle of Internal Friction (degrees) (φ')				
	20	25	30	35	40
	S= Average Friction Resistance on Pile Surface (psf)				
500	182	233	289	350	420
1000	364	466	577	700	839
1500	546	699	866	1050	1259
2000	728	933	1155	1400	1678
2500	910	1166	1443	1751	2098
3000	1092	1399	1732	2100	2517
3500	1274	1632	2021	2451	2937
4000	1456	1865	2309	2801	3356

5.7 LATERAL CAPACITY OF HELICAL PILES

Introduction

The primary function of a deep foundation is to resist axial loads. In some cases they will be subjected to horizontal or lateral loads. Lateral loads may be from wind, seismic events, live loads, water flow, etc. The resistance to lateral loads is in part a function of the near surface soil type and strength, and the effective projected area of the structure bearing against these soils. This section provides a summarized description of the methods and procedures available to determine the lateral capacity of helical piles/anchors in soil.

The analysis of deep foundations under lateral loading is complicated because the soil reaction (resistance) at any point along the shaft is a function of the deflection, which in turn is dependent on the soil resistance. Solving for the response of a deep foundation under lateral loading is one type of soil-structure interaction problem best suited for numerical methods on a computer. Square shaft (SS) helical piles/anchor do not provide any significant resistance to lateral loads. However, Round Shaft (RS) helical piles/anchor and HELICAL PULLDOWN® Micropiles can provide significant resistance to lateral loads depending on the soil conditions. Over the past 7 seven years, there has been considerable research done on the lateral capacity of grouted shaft helical piles – both with and without casing. Abdelghany & Naggar (2010) and Sharnouby & Naggar (2011) applied alternating cyclic lateral loads to helical piles of various configurations in an effort to simulate seismic conditions. Their research showed that helical piles with grouted shafts retain all their axial load capacity after being subjected to high displacement lateral load.

Lateral Resistance - Methods Used

Most helical piles/anchors have slender shafts [less than 3 inch (89 mm)] that offer limited resistance to lateral loads when applied to vertically installed shafts. Load tests have validated the concept that vertical pile foundations are capable of resisting lateral loads via shear and bending. Several methods are available to analyze the lateral capacity of foundations in soil including: 1) Finite Difference method; 2) Broms' Method (1964a) and (1964b); 3) Murthy (2003) Direct Method; and 4) Evans & Duncan (1982) Method as presented by Coduto (2001). Each of these methods may be applied to Round Shaft helical piles..

Lateral resistance can also be provided by passive earth pressure against the structural elements of the foundation. The resisting elements of the structure include the pile cap, grade beams and stem walls. The passive earth pressure against the structural elements can be calculated using the Rankine Method.

Battered or inclined helical piles/anchors can be used to resist lateral loads by assuming that the horizontal load on the structure is resisted by components of the axial load. The implicit assumption in this is that inclined foundations do not deflect laterally, which is not true. Therefore, it is better practice to use vertically installed helical piles/anchors to resist only vertical loads and inclined helical piles/anchors to resist only lateral loads. When inclined piles are required to resist both vertical and lateral loads, it is good practice to limit the pile inclination angle to less than 15°.

Friction resistance along the bottom of a footing, especially in the case of a continuous strip footing or large pile cap, can be significant. The friction component in a sandy soil is simply the structure's dead weight multiplied by the tangent of the angle of internal friction. In the case of clay, cohesion times the area of the footing may be used for the friction component. When battered piles are used to prevent lateral movement, the friction may be included in the computation. The designer is advised to use caution when using friction for lateral resistance. Some building codes do not permit friction resistance under pile supported footings and pile caps due to the possibility the soil will settle away from the footing or pile cap. Shrink-swell soils, compressible strata, and liquefiable soil can result in a void under footings and pile caps.

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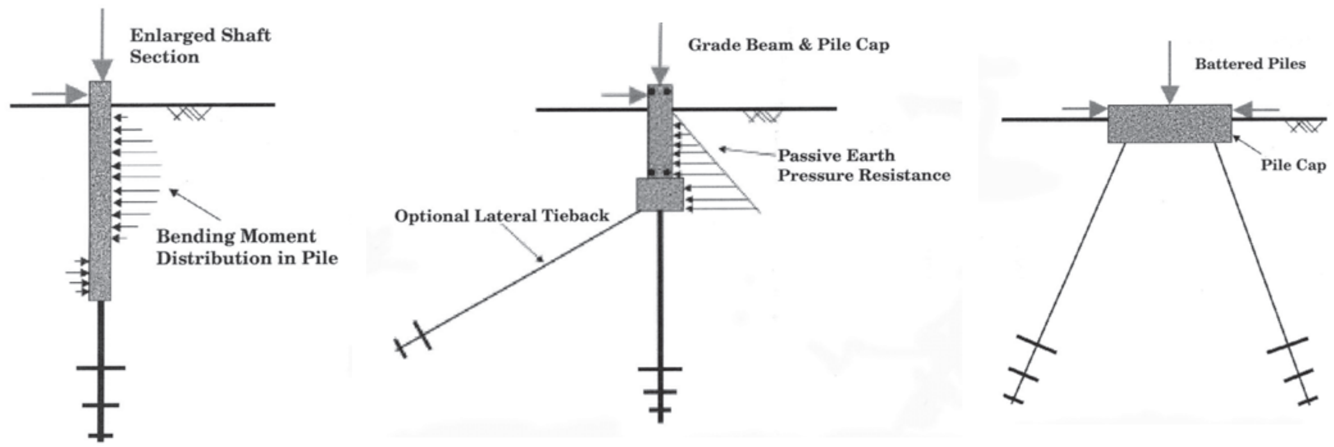

Figure 5-17. Lateral Resistance Methods

Finite Difference Method

Several computer programs, such as LPILEPLUS (ENSOFT, Austin, TX) are revisions of the COM624 program (Matlock and Reese) and its predecessor Beam-Column 28 (Matlock and Haliburton) that both use the p-y concept, i.e., soil resistance is a non-linear function of pile deflection, which was further developed by Poulos (1973). This method is versatile and provides a practical design method. This is made possible by the use of computers to solve the governing non-linear, fourth-order differential equation, which is explained in greater detail on page 5-20. Lateral load analysis software gives the designer the tools necessary to evaluate the force-deflection behavior of a helical pile/anchor embedded in soil.

Figures 5-18 and 5-19 are sample LPILE^{PLUS} plots of lateral shaft deflection and bending moment vs. depth where the top of the pile is fixed against rotation. From results like these, the designer can quickly determine the lateral response at various horizontal loads up to the structural limit of the pile, which is typically bending. Many geotechnical consultants use LPILE^{PLUS} or other soil-structure-interaction programs to predict soil-pile response to lateral loads.

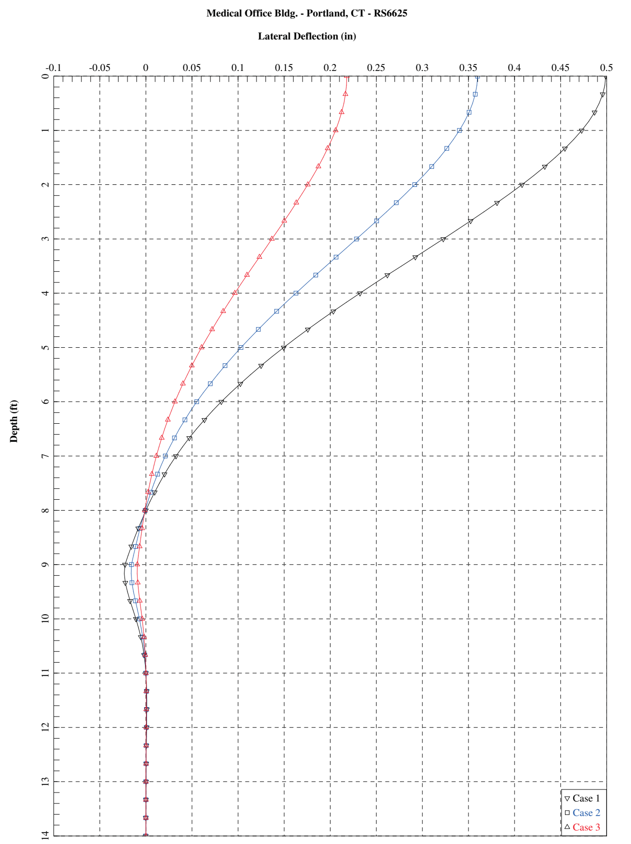

Figure 5-18. LPILE[PLUS] Sample Plot
Deflection vs Depth

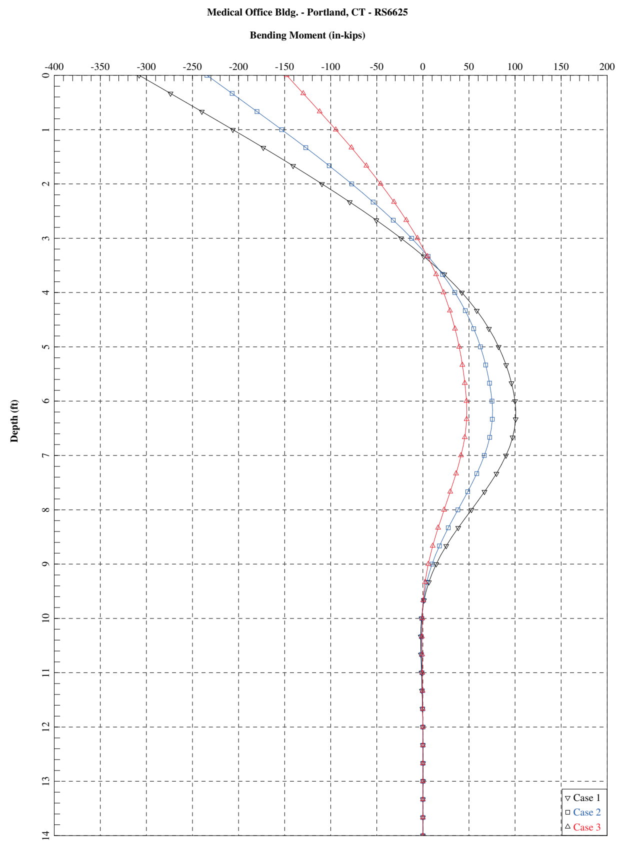

Figure 5-19. LPILE[PLUS] Sample Plot
Bending Moment vs Depth

Brom's (1964a & 1964b) Method

Broms' Method is best suited for applications where the top section of the helical pile/anchor/pile is a greater diameter than the bottom section. Enlarged top sections are commonly used to increase the lateral capacity of the foundation shaft. Design Example 8-13 in Section 8 gives an example of this. It uses Broms' method for short piers in cohesive soil. A "short" pier is one that is rigid enough that it will move in the direction the load is tending by rotation or translation. A "long" pier is one that the top will rotate or translate without moving the bottom of the foundation, i.e., a plastic hinge will form.

Broms developed lateral capacity methods for both short and long piles in cohesive and non-cohesive soil. Broms theorized that a short free-headed pier rotates about a center, above the lower end of the foundation, without substantial deformation along its axis. The resistance is the sum of the net of the earth pressures above and the passive earth pressure below the center of rotation. The end bearing influence or effect is neglected. Likewise, the passive earth pressure on the uppermost 1.5 diameters of shaft and the active earth pressure on the back of the pile are neglected.

Figure 5-20 is a reaction/shear/moment diagram that demonstrates the Broms theory for laterally loaded short piles in cohesive soils. A simple static solution of these diagrams will yield the required embedment depth and shaft diameter of the top section required to resist the specified lateral load. It is recommended the designer obtain and review Broms' technical papers (see References at the end of this section) to familiarize themselves with the various solution methods in both cohesive and non-cohesive soils. The Broms Method was probably the most widely used method prior to the finite difference and finite element methods used today and gives fair agreement with field results for short piles.

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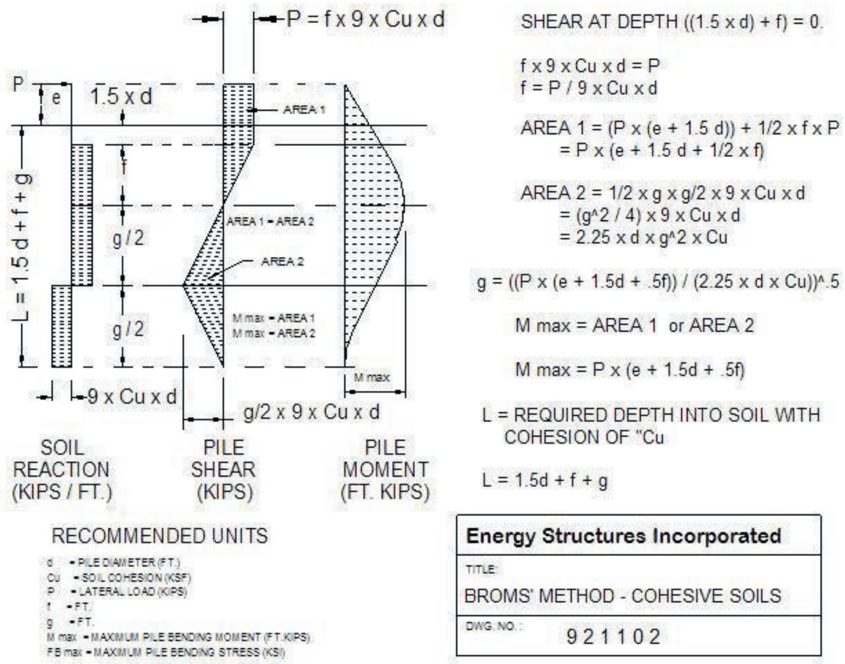

Figure 5-20. Broms' Method for Short Piles in Clay
(Energy Structures, Inc., 1994)

Lateral Capacity By Passive Earth Pressure

Passive earth pressure on the projected area of the pile cap, grade beam, or stem wall can be calculated by the Rankine (ca. 1857) method, which assumes no soil cohesion or wall-soil friction. One can use known or assumed soil parameters to determine the sum of the passive earth pressure minus the active earth pressure on the other side of the foundation as shown in Figure 5-21. The following are general equations to calculate active and passive pressures on a wall for the simple case on a frictionless vertical face and a horizontal ground surface. Equations 5-51 and 5-52 are Rankine equations for sand. Equations 5-53 and 5-54 are the addition of the cohesion for clay or cohesive soils. Three basic conditions are required for validity of the equations:

1. The soil material is homogenous.
2. Sufficient movement has occurred so shear strength on failure surface is completely mobilized.
3. Resisting element is vertical; resultant forces are horizontal.

$K_0 = 1\text{-}\sin \phi'$ **Equation 5-48**

$K_a = \tan^2 (45\text{-}\phi'/2)$ **Equation 5-49**

$K_p = \tan^2 (45+\phi'/2)$ **Equation 5-50**

For granular soil (sand):

$P_a = ½K_a\rho H^2$ **Equation 5-51**

$P_p = ½K_p\phi\rho H^2$ **Equation 5-52**

For cohesive soil (clay):

$P_a = ½K_a\rho H^2 – 2cH + 2c^2/\phi'\rho$ **Equation 5-53**

$Pp = ½K_p\rho H^2 + 2cH$ **Equation 5-54**

where: K_0 = Coefficient of earth pressure at rest
K_a = Coefficient of active earth pressure
K_p = Coefficient of passive earth pressure
H = Height of wall or resisting element
c = Cohesion
ϕ' = Effective stress friction angle of soil
P_a = Active earth pressure
ρ = Unit weight of soil

Equations 5-48 through 5-54 are from NAVFAC Design Manual DM7, Foundations and Earth Structures (see References at the end of this section).

Table 5-15 is a tabulation of the coefficient for at rest, active, and passive earth pressure for various soil types, relative densities and consistencies.

Table 5-15 Coefficients of Earth Pressure (Das, 1987)

Soil	K_0, Drained	K_0, Total	K_a, Total	K_p, Total
Clay, soft *	0.6	1	1	1
Clay, hard *	0.5	0.8	1	1
Sand, loose	0.6	0.53	0.2	3
Sand, dense	0.4	0.35	0.3	4.6
* Assume saturated clays				

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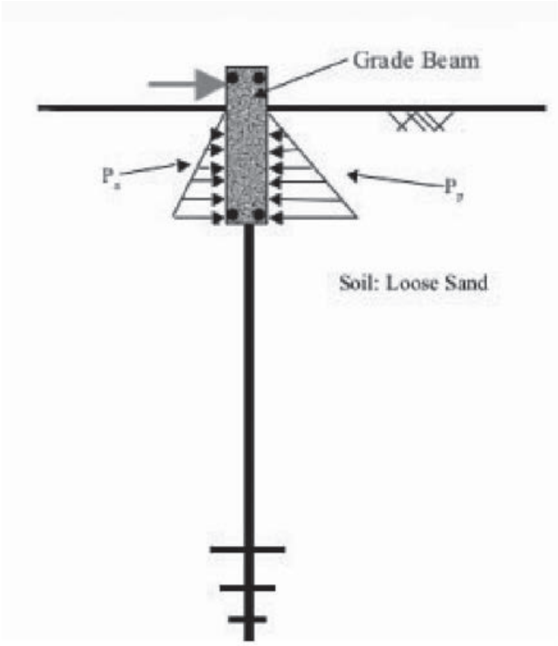

Figure 5-21. Earth Pressure on a Grade Beam

Using the Rankine solution may be an over-simplification of the problem but tends to be conservative since the height of the projected area of the footing or pile cap is not large and the cohesion term will generally be small. Design Example 8-15 in Section 8 illustrates the use of the Passive Resistance method to determine the lateral capacity of a foundation.

Battered CHANCE® Helical Piles/Anchors for Lateral Loading

Lateral loads are commonly resolved with battered helical piles and anchors. The method is to statically resolve the axial load capacity into its vertical and horizontal components. As stated earlier, it is best to use vertically installed helical piles and anchors to resist only vertical loads and battered helical piles and anchors to resist only lateral loads.

CHANCE® Helical Piles and Anchors and piles have been supplied to the seismic prone areas of the west coast of the United States and Canada for over 30 years for civil construction projects. In tension applications, they have been in service for over 50 years. They have been subjected to many earthquakes and aftershocks with good experience. Our helical pre-engineered products have been used far more extensively than any other manufacturer's helical product in these areas. To date, there have been no ill effects observed using battered helical piles and anchors in seismic areas. These foundations, both vertically installed and battered, have been subjected to several earthquakes of magnitude 7+ on the Richter scale with no adverse affects. Anecdotal evidence indicates the structures on helical piles experienced less earthquake-induced distress than their adjacent structures on other types of foundations. Their performances were documented anecdotally in technical literature, including the *Engineering News Record*.

Additional Comments

The lateral capacity of round shaft (Type RS) helical piles and anchors is greater than the square shaft (Type SS) helical anchors and piles because of the larger section size. Typical pipe diameters of 2-7/8" (73mm), 3-1/2" (89 mm) and 4-1/2" (114 mm) OD are used for CHANCE® Helical Piles. As shown in Design Example 8-13 in Section 8, enlarged shaft sections are used for certain applications. From a practical standpoint, the largest diameter helical pile available from Hubbell Power Systems, Inc. is 10-3/4" diameter, but larger shaft diameters are available on a project specific basis.

As previously noted, there are several other methods used to analyze the lateral capacity of the shaft of piles. Murthy (2003) also presented a direct method for evaluating the lateral behavior of battered (inclined) piles.

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5.8 BUCKLING/BRACING/SLENDERNESS CONSIDERATIONS

Introduction

Buckling of slender foundation elements is a common concern among designers and structural engineers. The literature shows that several researchers have addressed buckling of piles and micropiles over the years (Bjerrum 1957, Davisson 1963, Mascardi 1970, and Gouvenot 1975). Their results generally support the conclusion that buckling is likely to occur only in soils with very poor strength properties such as peat, very loose sands, and soft clay.

However, it cannot be inferred that buckling of a helical pile will never occur. Buckling of helical piles in soil is a complex problem best analyzed using numerical methods on a computer. It involves parameters such as the shaft section and elastic properties, coupling strength and stiffness, soil strength and stiffness, and the eccentricity of the applied load. This section presents a description of the procedures available to evaluate buckling of helical piles, and recommendations that aid the systematic performance of buckling analysis. Buckling of helical piles under compression loads, especially square shaft helical piles, may be important in three situations:

1. When a pile is relatively long (>20 feet [6 m]) and is installed through very soft clay into a very hard underlying layer and is end-bearing.
2. When a pile is installed in loose, saturated clean sand that undergoes liquefaction during an earthquake event.
3. When a pile is subject to excessive eccentric load without adequate bracing.

Bracing

Bracing of pile foundation elements is a common concern among designers and structural engineers, especially for helical piles and resistance piers with slender shafts. Section 1810.2.2 of the 2009 & 2012 International Building Code requires deep foundations to be braced to provide lateral stability in all directions. Bracing can be provided many different ways – including pile groups of three or more, alternate lines of piles spaced apart, and using slabs, footings, grade beams and other foundation elements to provide lateral stability. When CHANCE® Helical Piles and ATLAS RESISTANCE® Piers are used for foundation repair, the piers must be braced as per situation 3 above. The following figures show two methods that are often used to ensure adequate bracing is used.

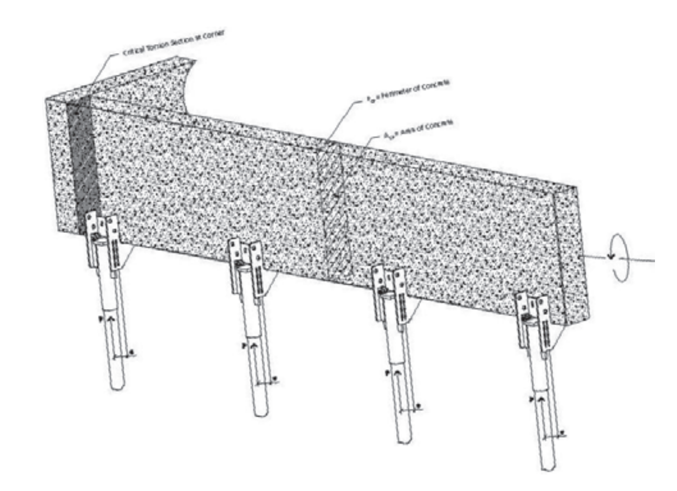

Figure 5-22

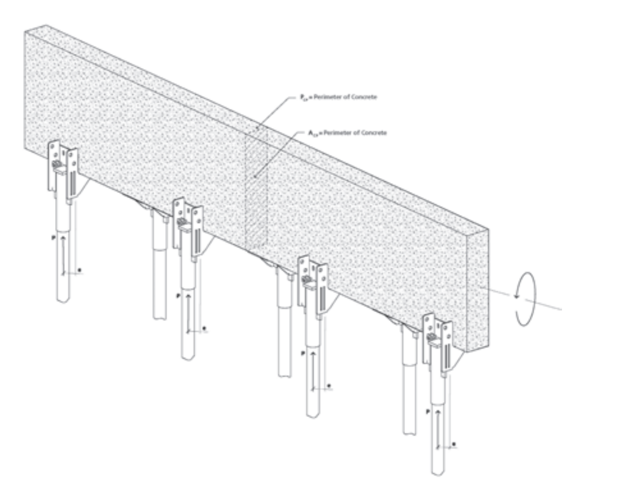

Figure 5-23

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Figure 5-22 on the left is a portion of a grade beam foundation underpinned with ATLAS RESISTANCE® Piers. The grade beam provides torsional stiffness based on its section properties and steel reinforcement. The 90° foundation element on the left end also provides torsional and shear stiffness. Figure 5-23 on the right is a portion of a long continuous grade beam foundation underpinned with ATLAS RESISTANCE® Piers. The piers are staggered and alternated both on the inside and outside, which provides bracing.

Buckling Background

Buckling of columns most often refers to the allowable compression load for a given unsupported length. The mathematician Leonhard Euler solved the question of critical compression load in the 18th century with a basic equation included in most strength of materials textbooks.

$$P_{crit} \quad \pi^2 EI/(KL_u)^2 \qquad \textbf{Equation 5-55}$$

	P_{crit}		$\pi^2EI/(KL_u)^2$
	E	=	Modulus of elasticity
where	I	=	Moment of inertia
	K	=	End condition parameter that depends on fixity
	L_u	=	Unsupported length

Most helical piles have slender shafts which can lead to very high slenderness ratios (Kl/r), depending on the length of the foundation shaft. This condition would be a concern if the helical piles were in air or water and subjected to a compressive load. For this case, the critical buckling load could be estimated using the well-known Euler equation above.

However, helical piles are not supported by air or water, but by soil. This is the reason helical piles can be loaded in compression well beyond the critical buckling loads predicted by Equation 5-55. As a practical guideline, soil with N_{60} SPT blow counts per ASTM D-1586 greater than 4 along the entire embedded length of the helical pile shaft has been found to provide adequate support to resist buckling - provided there are no horizontal (shear) loads or bending moments applied to the top of the foundation. Only the very weak soils are of practical concern. For soils with N_{60} values of 4 blows/ft or less, buckling calculations can be done by hand using the Davisson Method (1963) or by computer solution using the finite-difference technique as implemented in the LPILE^{PLUS} computer program (ENSOFT, Austin, TX). In addition, the engineers at Hubbell Power Systems, Inc. have developed a macro-based computer solution using the finite-element technique with the ANSYS® analysis software. If required, application engineers can provide project specific buckling calculations - given sufficient data relating to the applied loads and the soil profile. If you need engineering assistance, please contact your CHANCE® Distributor in your area. Contact information for CHANCE® Distributors can be found at www.abchance.com. These professionals will help you to collect the data required to perform a buckling analysis. The distributor will either send this data to Hubbell Power Systems, Inc. for a buckling analysis or provide this service themselves.

Buckling/Lateral Stability per International Building Code (IBC) Requirements

IBC 2009 Section 1810.2.1 - Lateral Support states that any soil other than fluid soil shall be deemed to afford sufficient lateral support to prevent buckling of deep foundation elements in accordance with accepted engineering practice and the applicable provisions of this code. Per IBC 2006 Section 1808.2.9.2 & IBC 2009 Section 1810.2.1, pier/piles driven into firm ground can be considered fixed and laterally supported at 5 feet below the ground surface and in soft material at 10 feet below the ground surface. The IBC does not specifically define fluid, soft, and firm soil. To remedy this, ICC-ES Acceptance Criteria AC358 defined these soil terms as follows:

Firm soils are defined as any soil with a Standard Penetration Test blow count of five or greater.

Soft soils are defined as any soil with a Standard Penetration Test blow count greater than zero and less than five.

Fluid soils are defined as any soil with a Standard Penetration Test blow count of zero [weight of hammer (WOH) or weight of rods (WOR).

Therefore, one method to check the effects of buckling and lateral stability of helical piles and resistance piers is to assume the depth to fixity is either 5 feet in firm soil, or 10 feet in soft soil. The corresponding axial compression capacity of the pile shaft is determined based on either 5 feet or 10 feet of unsupported length. This is the method used to determine the nominal, LRFD design, and ASD allowable compression strengths of the helical pile product families provided in Section 7 of this manual.

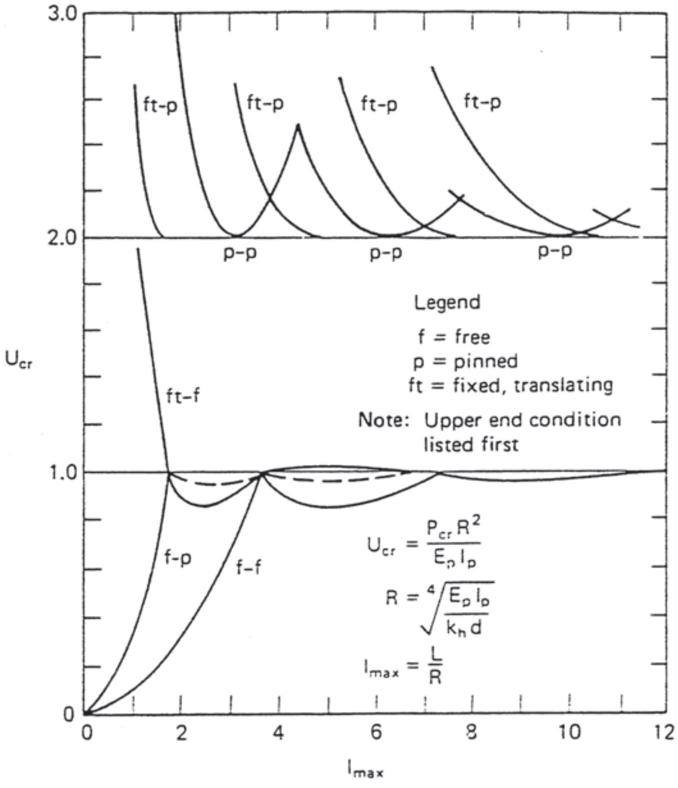

Figure 5-24 Poulos and Davis (1980)

Buckling Analysis by Davisson (1963) Method

A number of solutions have been developed for various combinations of pile head and tip boundary conditions and for the cases of constant modulus of sub grade reaction (kh) with depth. One of these solutions is the Davisson (1963) Method as described below. Solutions for various boundary conditions are presented by Davisson in Figure 5-24. The axial load is assumed to be constant in the pile – that is no load transfer due to skin friction occurs and the pile initially is perfectly straight. The solutions shown in Figure 5-24 are in dimensionless form, as a plot of U_{cr} versus I_{max}.

where $U_{cr} = P_{cr}R^2/E_pI_p$ or $P_{cr} = U_{cr}E_pI_p/R^2$ **Equation 5-56**

where $R = \sqrt[4]{E_pI_p/k_hd}$ **Equation 5-57**

where I_{max} = L/R **Equation 5-58**

P_{cr} = Critical buckling load

E_p = Modulus of elasticity of foundation shaft

I_p	=	Moment of inertia of foundation shaft
K_h	=	Modulus of sub grade reaction
d	=	Foundation shaft diameter
L	=	Foundation shaft length over which k_h is taken as constant
U_{cr}	=	Dimensionless ratio

By assuming a constant modulus of sub grade reaction (kh) for a given soil profile to determine R, and using Figure 5-24 to determine U_{cr}, Equation 5-56 can be solved for the critical buckling load. Typical values for k_h are shown in Table 5-16.

Table 5-16. Modulus of Sub Grade Reaction - Typical Values

Soil Description	odulus of Subgrade Reaction (K_h) (pci)
Very soft clay	15 - 20
Soft clay	30 - 75
Loose sand	20

Figure 5-24 shows that the boundary conditions at the pile head and tip exert a controlling influence on U_{cr}, with the lowest buckling loads occurring for piles with free (unrestrained) ends. Design Example 8-16 in Section 8 illustrates the use of the Davisson (1968) method to determine the critical buckling load.

Another way to determine the buckling load of a helical pile in soil is to model it based on the classical Winkler (mathematician, circa 1867) concept of a beam-column on an elastic foundation. The finite difference technique can then be used to solve the governing differential equation for successively greater loads until, at or near the buckling load, failure to converge to a solution occurs. The derivation for the differential equation for the beam-column on an elastic foundation was given by Hetenyi (1946). The assumption is made that a shaft on an elastic foundation is subjected not only to lateral loading, but also to compressive force acting at the center of the gravity of the end cross-sections of the shaft, leading to the differential equation:

$$EI(d^4y/dx^4) + Q(d^2y/dx^2) + E_s y = 0$$

where

y	=	Lateral deflection of the shaft at a point x along the length of the shaft
x	=	Distance along the axis, i.e., along the shaft
E	=	Flexural rigidity of the foundation shaft
Q	=	Axial compressive load on the helical pile
$E_s y$	=	Soil reaction per unit length
E_s	=	Secant modulus of the soil response curve

The first term of the equation corresponds to the equation for beams subject to transverse loading. The second term represents the effect of the axial compressive load. The third term represents the effect of the reaction from the soil. For soil properties varying with depth, it is convenient to solve this equation using numerical procedures such as the finite element or finite difference methods. Reese, et al. (1997) outlines the process to solve Equation 5-59 using a finite difference approach. Several computer programs are commercially available that are applicable to piles subject to axial and lateral loads as well as bending moments. Such programs allow the introduction of soil and foundation shaft properties that vary with depth, and can be used advantageously for design of helical piles and micropiles subject to centered or eccentric loads.

To define the critical load for a particular structure using the finite difference technique, it is necessary to analyze the structure under successively increasing loads. This is necessary because the solution algorithm becomes unstable at loads above the critical. This instability may be seen as a convergence to a physically illogical configuration or failure to converge to any solution. Since physically illogical configurations are not always easily recognized, it is best to build up a context of correct solutions at low loads with which any new solution can be

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compared. Design Example 8-17 in Section 8 illustrates the use of the Finite Difference method to determine the critical buckling load.

Buckling Analysis by Finite Elements

Hubbell Power Systems, Inc. has developed a design tool, integrated with ANSYS® finite element software, to determine the load response and buckling of helical piles. The method uses a limited non-linear model of the soil to simulate soil resistance response without increasing the solution time inherent in a full nonlinear model. The model is still more sophisticated than a simple elastic foundation model, and allows for different soil layers and types.

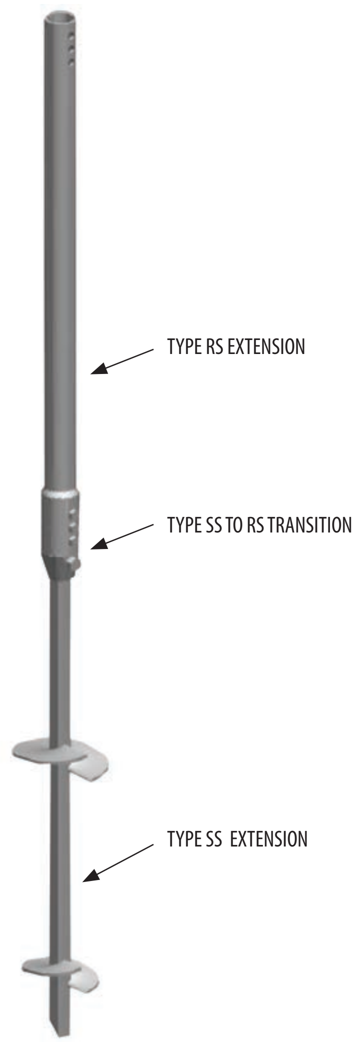

Figure 5-25 Type SS to RS Combination Pile

The helical pile components are modeled as 3D beam elements assumed to have elastic response. Couplings are modeled from actual test data, which includes an initial zero stiffness, elastic/rotation stiffness and a final failed condition – which includes some residual stiffness. Macros are used to create soil property data sets, helical pile component libraries, and load options with end conditions at the pile head.

After the helical pile has been configured and the soil and load conditions specified, the macros increment the load, solve for the current load and update the lateral resistance based on the lateral deflection. After each solution, the ANSYS® post-processor extracts the lateral deflection and recalculates the lateral stiffness of the soil for each element. The macro then restarts the analysis for the next load increment. This incremental process continues until buckling occurs. Various outputs such as deflection and bending moment plots can be generated from the results. Design Example 8-18 in Section 8 illustrates the use of the Finite Element method to determine the critical buckling load.

Practical Considerations – Buckling

As stated previously, where soft and/or loose soils (SPT N_{60} blow count ≤ 4) overlie the bearing stratum, the possibility of shaft buckling must be considered. Buckling also becomes a potential limiting factor where lateral loads (bending and shear) are present in combination with compressive loads. Factors that determine the buckling load include the helical pile shaft diameter, length, flexural stiffness and strength, the soil stiffness and strength, any lateral shear and/or moment applied at the pile head, and pile head fixity conditions (fixed, pinned, free, etc.). In addition, all extendable helical piles have couplings or joints used to connect succeeding sections together in order to install the helix plates into bearing soil. Bolted couplings or joints have a certain amount of rotational tolerance. This means the joint initially has no stiffness until it has rotated enough to act as a rigid element. This is analogous to saying the coupling or joint acts as a pin connection until it has rotated a specific amount, after which it acts as a rigid element with some flexural stiffness.

Concern about slender shafts and joint stiffness, along with the fact that helical piles are routinely installed in soils with poor strength; are some of the reasons why helical piles are often installed with grouted shafts (helical pulldown piles) and are available with larger diameter pipe shafts (Type RS). Pipe shaft helical piles have better buckling resistance than plain square shaft (Type SS) because they have greater section modulus (flexural resistance), plus they have larger lateral dimensions, which means they have greater resistance to lateral deflection in soil. See the specifications section of the helical pile product family pages in Section 7 for the section properties and dimensions of both Type SS and RS helical piles/anchors.

Type SS helical piles/anchors provide the most efficient capacity-to-torque relationship (see Section 6, Installation Methodology). Type RS helical piles/anchors provide lateral capacity and better buckling resistance. A good compromise to address buckling in soft/loose soils is to use helical combination piles, or "combo piles" for short. A combo pile consists of Type SS square shaft material for the lead section and Type RS pipe shaft material for the extension sections (see Figure 5-25). The combo pile provides the advantages of both Type SS and RS material, which enables the helical pile/anchor to penetrate dense/hard soils, while at the same time provide a larger shaft section in the soft/loose soils above the bearing strata. See Section 7 for more information on combo piles.

The HELICAL PULLDOWN® Micropile is a method for constructing a grout column around the shaft of either a Type SS (square shaft) or RS (round shaft) helical pile installed in soft/loose soil. The installation process displaces soil around the central steel shaft and replaces it with a gravity fed, neat cement grout mixture. Upon curing, the grout forms a column that increases the section modulus of the pile shaft to the point that buckling is not the limiting condition. In addition to buckling resistance, the grout column increases axial load capacity due to skin friction or adhesion along the shaft; plus the load/deflection response of the helical pile is stiffer. See Section 7 for more information on CHANCE HELICAL PULLDOWN® Micropiles.

CHANCE HELICAL PULLDOWN® Micropiles cannot be installed in every soil condition. To date, grouted shaft helical piles have been successfully installed in overburden soil with SPT blow counts greater than 10 blows/ft. In those cases, the grouted shaft is being used to develop greater load capacity and a stiffer response, not necessarily to prevent buckling. Contractors have successfully installed pulldown micropiles in glacial tills (SPT N_{60} > 50) using special soil displacement methods. Increasingly dense soil makes installation more difficult for the displacement element, which has to force soil laterally outward away from the central steel shaft.

5.9 HELICAL PILE DEFLECTION AT WORKING LOAD

Most of the discussion thus far has focused on evaluating the ultimate load capacity of helical piles/anchors in axial compression or tension. This is considered as the Load Limit State and gives the upper bound on the load capacity. The displacements of the pile/anchor at this load state will be very large (> 2 inches [51 mm]) and technically the pile/anchor cannot sustain additional load but the deflection just keeps increasing. However, it is also of great interest to most engineers to consider the behavior of a helical pile/anchor at a lower working load or Serviceability State which will be well below the Load Limit State.

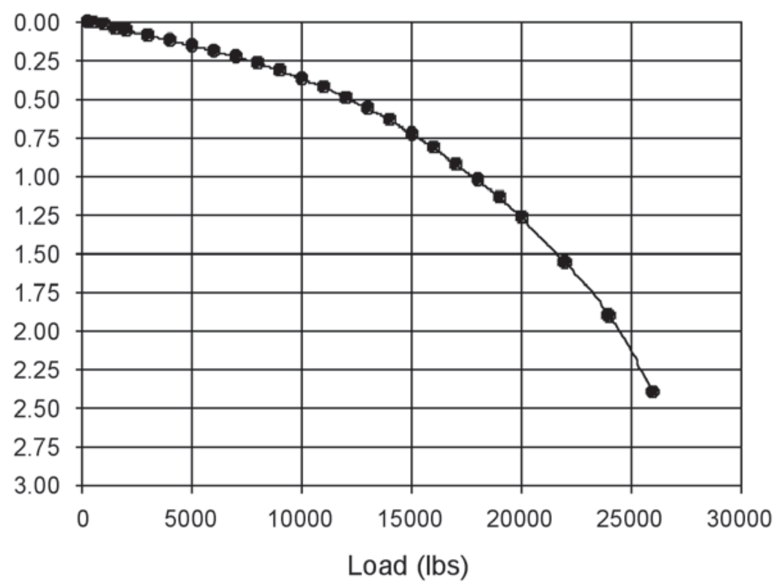

Figure 5-26

We can consider a typical Load-Displacement curve as shown above. This plot is the test results of a 1.5 in. x 1.5 in. square-shaft helical anchor with a single 12 in. helix installed to a depth of 10 ft. in a medium dense silty sand. The test was performed in tension. According to the IBC, the Ultimate Capacity may be taken as the load producing a net displacement of 10% of the helix

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diameter or in this case the load at 1.20 in. which is 19,500 lbs. It is obvious that in this case, as in most cases, the anchor can actually take more load, up to as much as 20% of the helix diameter.

Using a ASD Factor of Safety of 2.0, the working load for this anchors would be equal to 19,500 lbs/2.0 = 9,750 lbs. Because the load-displacement curve of most helical piles/anchors is generally nonlinear it would be expected that the displacement at the working load would be less than ½ of the displacement at 1.20 in. In this case, the displacement at the working load of 9,750 lbs is on the order of 0.36 in. Using a lower Factor of Safety gives a higher displacement. For example if a Factor of Safety of 1.5 is used, the working load becomes 19,500 lbs/1.5 = 13,000 lbs and the displacement corresponding to this load is on the order of 0.55 in.

Based on a review of a number of tests performed on single-helix pile/anchors in Colorado, Cherry and Perko (2012) recently suggested that for many anchors/piles, the displacement at the working loads (F.S. = 2) averaged about 0.25 in. Additional work is needed to determine how this may vary for multi-helix piles/anchors and if other soils show different behavior.

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